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TECHNICAL REPORT S-68-8

EFFECT OF SHALLOW BURIAL ON LOAD-DISPLACEMENT-TIME RESPONSE OF SQUARE FOOTINGS IN CLAY UNDER IMPULSIVE LOADING

by

E. B. Perry

November 1968

Sponsored by

Defense Atomic Support Agency

Conducted by

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

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TECHNICAL REPORT S-68-3

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NWER Subtask RSS3210010C

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ARMY-MRC VICKSBURG, MISS.

ABSTRACT

The advent of nuclear weapons has resulted in the need for shallow-buried protective structures capable of surviving ground shock and associated loadings induced by high-intensity nuclear air-blasts The design of such structures requires a knowledge of the behavior of their foundations under rapidly applied transient loads.

The purpose of this investigation was to determine if nondimensional relations developed previously by the application of similitude theory and dimensional analysis for dynamically loaded surface footings on clay also hold for dynamically loaded footings at shallow depth of burial and to determine the effect of shallow burial on the footing response.

The results of the investigation showed that the nondimensional relations developed for surface footings on clay also hold for footings at shallow depth of burial. The effect of shallow burial on the response of dynamically loaded footings in clay is small and can be considered negligible for design purposes.

PREFACE

This investigation was conducted by the U. S. Army Engineer Waterways Experiment Station (WES) under the sponsorship of the Defense Atomic Support Agency (Nuclear Weapons Effects Research Subtask RSS3210010C). The testing was conducted during the period January to April 1967. The tests were conducted and the report prepared under the supervision of Mr. P. F. Hadala and the general direction of Messrs. W. J. Turnbull, A. A. Maxwell, R. W. Cunny, and J. G. Jackson, Jr., of the Soils Division.

The test program was planned by Mr. E. B. Perry, and executed by Mr. R. C. Sloan. Messrs. B. F. Wright, B. C. Palmertree, A. G. Reno, and P. L. Marsicano assisted in the conduct of the tests and data reduction.

This report was prepared by Mr. Perry, and the material contained herein was submitted as a thesis in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering to Mississippi State University, State College, Mississippi.

COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE, were Directors of the WES during this investigation. Mr. J. B. Tiffany was Technical Director.

CONTENTS

TABLES

FIGURES

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows.

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
pounds	0.45359237	kilograms
kips	453.59237	kilograms
kips per square foot	4,822.4	kilograms per square meter
pounds per cubic foot	16.018	kilograms per cubic meter
inches per minute	25.4	millimeters per minute
inches per second	25.4	millimeters per second
inches per second per second	25.4	millimeters per second per second

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The advent of nuclear weapons has resulted in the need for shallow-buried protective structures capable of surviving ground shock and associated loadings induced by high-intensity nuclear air-blasts. The design of such structures requires a knowledge of the behavior of their foundations under rapidly applied transient loads.

In order to assist in the development of foundation design criteria for protective structures, a study of the dynamic bearing capacity of soils was initiated at the U. S. Army Engineer Waterways Experiment Station (WES). The purpose of this study was to develop the relations between blast-type time histories for footings and the resulting footing displacement. Small-scale footing tests were conducted using a loading machine that delivered controlled dynamic load pulses to footings in soil specimens contained in large movable carts (References 1-6).

In the WES studies, small-scale surface footing tests were conducted on sand to investigate the mode of failure and soil motions during the displacement of a dynamically loaded surface footing (Reference 2). The results of these tests indicated that the behavior patterns under dynamic loads were quite different from conventional

static bearing capacity failure patterns. Because the dynamic footing test data did not appear to fit classical failure theories, a new approach to the problem of predicting the displacement of dynamically loaded footings was undertaken. A prototype system that would be suitable for eventual full-scale field testing was chosen. The system consisted of a central surface footing on clay supporting a blast-loaded structure. Models of the postulated prototype were developed using the principles of similitude and tested using the dynamic loading machine and compacted clay specimens in mobile soil specimen carts.

The analysis of the results of the dynamic tests using nondimensional parameters yielded three useful relations: maximum footing load reaction as a function of maximum displacement, maximum displacement as a function of the dynamic load applied to the roof of the structure, and time to maximum displacement as a function of the dynamic load applied to the roof of the structure (Reference 3). The validity of these nondimensional relations was established in another study that showed that they were independent of the model scaling relations (Reference 4).

A simplified model of a field test structure was developed, and verified by comparison of the results of tests of the small-scale model with the results of tests of the actual prototype (Reference 6).

1.2 PURPOSES OF THIS INVESTIGATION

The purposes of this investigation were to determine if the form of the nondimensional relations developed previously for dynamically loaded surface footings on clay also holds for dynamically loaded footings at shallow depth of burial (depth of burial equal to one footing width), and to determine the effect of shallow depths of burial on the three nondimensional relations developed for surface footings.

1.3 SCOPE OF THIS INVESTIGATION

The test program was limited to a study of the response of square footings to dynamic axial loads of finite duration at a depth of burial equal to one footing width. One static and five dynamic footing tests were conducted on 4.5-inch[1]-wide square footings in a highly plastic, nearly saturated, compacted clay that had a static shear strength of approximately 1.10 kips per square foot. A similar series of tests was also conducted on 8-inch-wide footings in clay that had a static shear strength of 1.45 kips per square foot.

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[1] A table of factors for converting British units of measurement to metric units is presented on page 8.

CHAPTER 2

REVIEW OF PREVIOUS RESEARCH

2.1 STATIC FOOTING RESPONSE IN CLAY

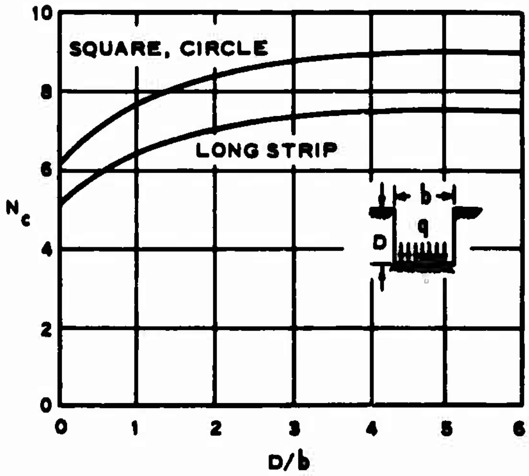
There is considerable published literature concerning the bearing capacity and settlement of foundations in clay. A review of small-scale footing studies covering the literature until 1960 is presented in Reference 7. Some of the more recent research is summarized in References 8 and 9.

This review is limited to research concerning the effect of depth of burial on the response of footings in clay. A solution is given in Reference 10 to obtain the bearing capacity of foundations in clay; this solution was based on laboratory footing tests, theoretical solutions to the bearing capacity problem for surface and deep footings, and observations of full-scale foundation failures. This solution (see Figure 2.1) for square footings at a depth of burial equal to one footing width, as used in this investigation, shows an increase in bearing capacity of approximately 25 percent over the bearing capacity of surface footings. This relation, however, is based on a rather limited number of laboratory tests conducted in remolded London clay (Reference 11). In these tests the effect of depth of burial was studied on 0.5-inch-diameter footings, with two tests each conducted at depths of burial equal to 0.5 and 4 footing

diameters. The tests utilized constant rates of penetration of 0.2 to 1.2 inches per minute with a typical test lasting between 5 and 10 minutes. The shear strength of the soil was determined by unconfined compression tests and was found to range from 0.15 to 0.45 kip per square foot.

2.2 DYNAMIC FOOTING RESPONSE IN CLAY

Research concerning the dynamic response of footings in clay is summarized in References 12 and 13. To the author's knowledge, there has been no research concerning the effects of depth of burial on the response of footings in clay under dynamic loading.



D/b	N_c SQUARE, CIRCLE	N_c LONG STRIP
0.00	6.2	5.2
0.25	6.7	5.6
0.50	7.1	5.9
0.75	7.4	6.2
1.00	7.7	6.4
1.50	8.1	6.6
2.00	8.4	7.0
2.50	8.6	7.2
3.00	8.8	7.4
4.00	9.0	7.5
5.00	9.0	7.5

$$q = c \cdot N_c + \gamma D$$

WHERE
q = BEARING CAPACITY
c = COHESION
N_c = BEARING CAPACITY FACTOR
γ = UNIT WEIGHT OF SOIL
D = DEPTH OF BURIAL
b = FOOTING WIDTH

Figure 2.1 Skempton's solution for bearing capacity.

CHAPTER 3

FOOTING TESTS

3.1 DEVELOPMENT OF NONDIMENSIONAL RELATIONS

The application of similitude theory and dimensional analysis to footing tests is reported in Reference 3. The variables that were developed and are believed to significantly affect the displacement response of the test system in this investigation, as shown in Figure 3.1, are listed in the following tabulation:

Variable	Definition	Units
z	Displacement of footing	L
b	Footing width	L
D	Depth of burial	L
P	Applied dynamic column load	F
t_o	Total pulse time	T
t	Other characteristic time	T
τ_f	Shear strength of soil	FL^{-2}
ρ	Mass density of soil	FT^2L^{-4}
m	Mass of load column of dynamic loading machine	FT^2L^{-1}

The nine variables can be rewritten in terms of six dimensionless parameters as follows:

$$\frac{z}{b} = f\left(\frac{D}{b}, \frac{t}{t_o}, \frac{P}{b^2\tau_f}, \frac{m}{b^3\rho}, \frac{Pt^2}{bm}\right) \tag{3.1}$$

where

$\frac{z}{b}$ = displacement parameter

$\frac{D}{b}$ = depth of burial parameter

$\frac{t}{t_o}$ = time parameter

$\frac{P}{b^2 \tau_f}$ = ratio of applied live-load force to soil-resistance force, or strength parameter

$\frac{m}{b^3 \rho}$ = ratio of structural mass to soil mass, or mass parameter

$\frac{Pt^2}{bm}$ = ratio of applied live-load force to inertia force, or inertia parameter

From the free-body diagram in Figure 3.1, the relation between the load $R(t)$ delivered to the footing and the dynamic driving force $P(t)$ is given as

$$P(t) - R(t) = m(a - g) \tag{3.2}$$

where

$P(t)$ = dynamic column load

$R(t)$ = footing reaction

m = mass of load column

a = acceleration of system

g = gravitational acceleration

Since the masses of the footings tested were small, $R(t)$ essentially represented the soil-resistance force opposing footing

displacement. This can be expressed in dimensionless form as

$$\frac{R}{b^2 \tau_f} = \text{ratio of average stress on base of footing to soil shearing resistance, or resistance parameter}$$

The analysis of static and dynamic surface footing tests in Reference 3 yielded the dimensionless footing reaction-displacement relation shown in Figure 3.2. The second nondimensional relation for surface footings, involving the maximum dynamic column load and the maximum displacement, is shown in Figure 3.3. The third dimensionless relation, between maximum applied dynamic column load and time to maximum displacement for the surface footings, is shown in Figure 3.4. The validity of these nondimensional relations was established by additional tests (Reference 4), and their application in predicting the response of a footing during a field test is reported in Reference 6.

3.2 PLAN OF TESTS

A test program (see Table 3.1) of 10 dynamic footing tests at depths equal to one footing width was planned to determine if the nondimensional relations developed for dynamically loaded surface footings on clay were applicable at shallow depth of burial and to evaluate the effects of the shallow burial on footing response. The maximum dynamic input and input rise times, hold times, and total pulse times, shown in Table 3.2, were determined from the idealized

dynamic column load-time curves in Appendix D. The description of the soil and soil specimen construction is given in Appendix A. The test system, the dynamic and static test procedures, and the soil sampling and testing procedures employed are described in Appendix B.

3.3 TEST RESULTS

The results of soil testing during specimen construction and unconfined compression and unconsolidated-undrained triaxial tests on undisturbed block samples taken from the soil carts are given in Appendix C. Detailed results of the footing tests, given in Appendix D, show computed column load, column acceleration, load cell reaction, and average footing displacement time histories for the dynamic tests (Figures D.1 to D.10). Load cell reaction versus average footing displacement for dynamic and static tests is also given (see Figures D.11 and D.12). The results of the footing tests are summarized in Table 3.2.

3.4 COMPARISON OF NONDIMENSIONAL RELATIONS

The resistance parameter versus the displacement parameter is shown in Figure 3.5. The dotted band is the range of the data for dynamic surface footing tests. The points represent the values of $\frac{R_{max}}{b^2\tau_f}$ and $\frac{z_{max}}{b}$ for the 10 dynamic footing tests at shallow depth of burial. Eight of the 10 data points fall within the band, with the

remaining two falling slightly above the band. The static resistance versus displacement curves are also shown in Figure 3.5. The hatched band represents the range of the data for static surface footing tests. The results of the static tests at shallow depth of burial fall within the band.

The relation between the strength parameter and displacement parameter is shown in Figure 3.6. Five of the 10 data points for the shallow depth of burial fall within the range of values for the surface footings. Of the remaining five points, four are slightly above the range and one at a relatively small displacement is just below the range.

The strength parameter versus inertia parameter is given in Figure 3.7. Seven of the 10 data points fall within the range of values, with the three remaining points falling close to the range.

3.5 EFFECT OF SHALLOW DEPTH OF BURIAL

The form of the nondimensional relations developed for dynamically loaded surface footings on clay also holds for dynamically loaded footings at shallow depth of burial. The effect of shallow depth of burial on the nondimensional relations is small and can be considered negligible for design purposes.

TABLE 3.1 TEST PLAN - INDEPENDENT PARAMETERS FOR FOOTING TESTS

Test No.	Test Location	Plate Width	Estimated Shear Strength τ_f	Estimated Wet Unit Weight γ_m	Maximum Dynamic Input P_{max}	Input Rise Time t_r	Input Hold Time t_h	Total Pulse Time t_o
		in.	kips/sq ft	pcf	kips	msec	msec	msec
58-1	2+00	8.0	2.00	115	8.00	8	0	389
58-2	3+50				3.30	9	0	425
58-3	5+00				8.00	8	0	389
58-4	6+50				6.52	8	0	410
58-5	8+00				6.52	8	0	410
58-6	10+00				Static Test			
59-1	2+00	4.5	1.13	115	2.30	5	0	260
59-2	3+50				0.88	7	0	320
59-3	5+00				1.74	6	0	280
59-4	6+50				1.42	6	0	300
59-5	8+00				1.16	6	0	310
59-6	10+00				Static Test			

TABLE 3.2 TEST RESULTS - CONTROLLED AND DEPENDENT PARAMETERS FOR FOOTING TESTS

Controlled Parameters

Test No.	Test Location	Plate Width	Estimated Shear Strength τ_f*	Estimated Wet Unit Weight γ_m**	Maximum Dynamic Input P_{max}	Input Rise Time t_r	Input Hold Time t_h	Total Pulse Time t_o
		in.	kips/sq ft	pcf	kips	msec	msec	msec
58-1	2+00	8.0	1.60	118.4	8.33	4	0	385
58-2	3+50	↓	1.30	118.8	2.95	5	0	405
58-3	5+00	↓	1.50	117.9	7.96	5	0	385
58-4	6+50	↓	1.25	117.6	6.23	4	0	330
58-5	8+00	↓	1.40	117.6	5.88	4	0	325
58-6	10+00	↓	1.60	117.0	← Static Test →			
59-1	2+00	4.5	1.05	116.9	2.32	5	0	240
59-2	3+50	↓	1.10	117.9	0.75	4	32	300
59-3	5+00	↓	1.05	116.5	1.67	6	0	275
59-4	6+50	↓	1.00	117.9	1.35	4	0	310
59-5	8+00	↓	1.05	116.6	1.13	6	0	300
59-6	10+00	↓	1.10	116.3	← Static tests →			

Dependent Parameters

Test No.	Maximum Load cell Reaction R_{max}	Time to R_{max} $t(R_{max})$	Maximum Plate Displacement z_{max}	Time to z_{max} $t(z_{max})$	Final Plate Displacement z_{final}	Computed Maximum Velocity V_{max}	Time to V_{max} $t(V_{max})$	Maximum Positive Acceleration a+	Maximum Negative Acceleration a-
	kips	msec	in.	msec	in.	ips	msec	g	g
58-1	9.06	29	2.06	45	1.54	87.8	16	36.7	-11.1
58-2	3.75	11	0.08	30	0.03	12.9	7	11.0	-5.0
58-3	8.89	29	2.10	47	1.83	82.3	17	36.9	-10.0
58-4	6.83	24	0.90	36	0.67	48.0	14	27.5	-8.1
58-5	6.53	22	0.71	36	0.51	40.5	13	26.2	-7.3
59-1	2.36	46	2.09	79	1.90	51.9	30	12.0	-3.8
59-2	1.18	35	0.22	50	0.16	7.5	23	2.7	-1.4
59-3	1.98	51	1.02	68	0.89	28.4	30	7.8	-2.9
59-4	1.61	46	0.79	61	0.67	22.8	25	6.8	-2.4
59-5	1.47	40	0.41	51	0.31	14.6	25	4.4	-2.0

* τ_f = average UC shear strength for cart $\times \dfrac{\text{vane resistance near plate}}{\text{average vane resistance for cart}}$.

** γ_m = average UC wet unit weight for cart $\times \dfrac{\text{posttest wet unit weight near test}}{\text{average posttest wet unit weight for cart}}$.

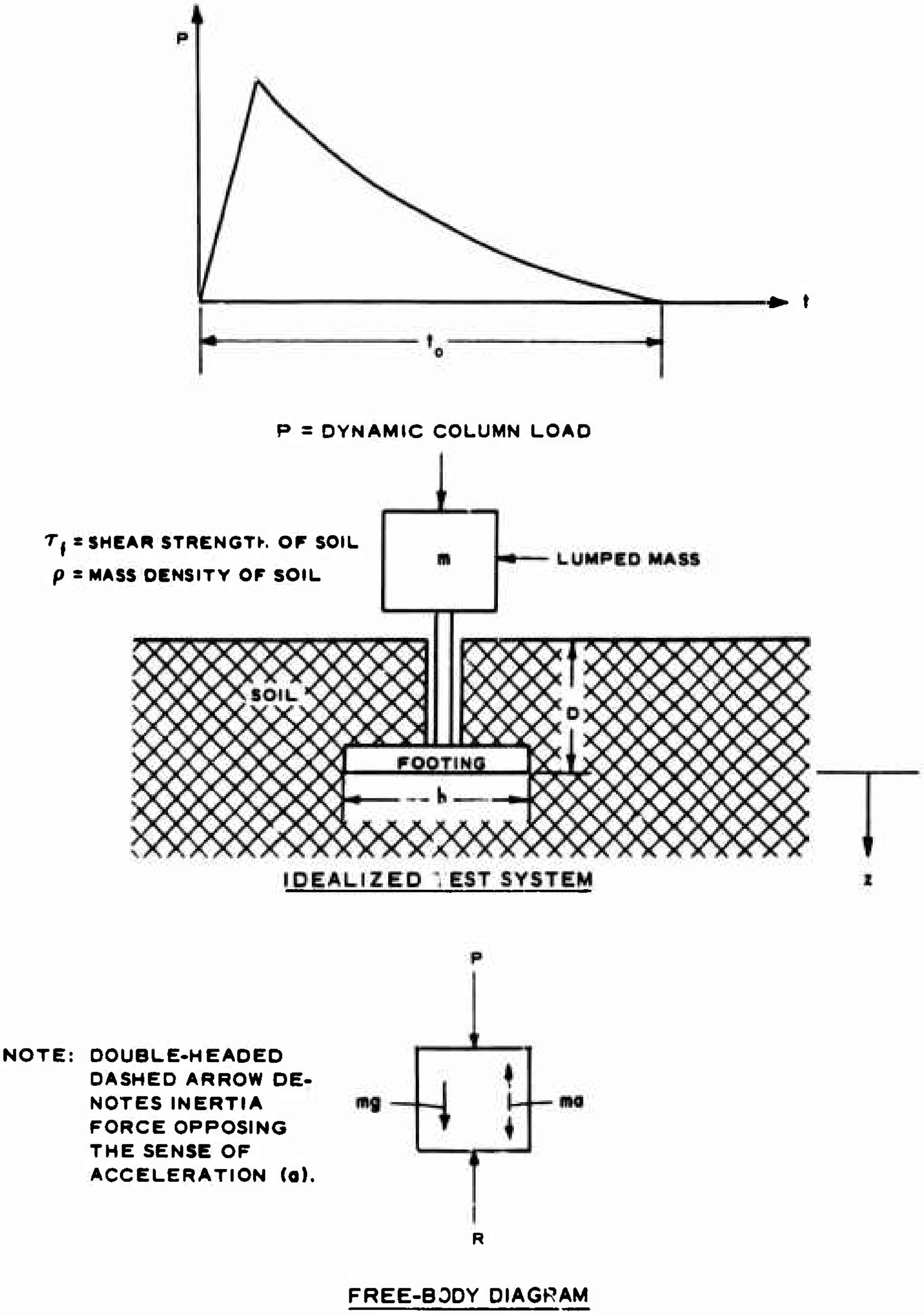
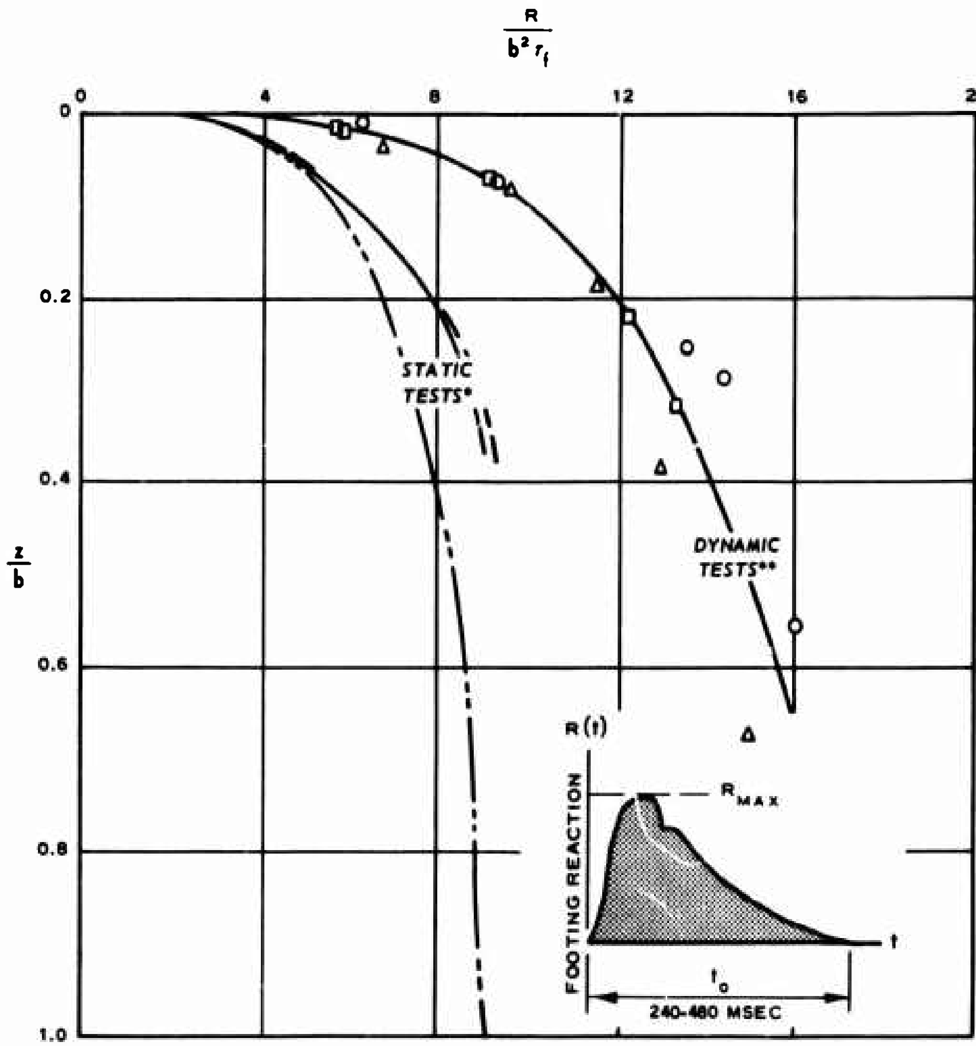


Figure 3.1 Idealized test system and free-body diagram.



LEGEND

SYMBOL	CART NO.	FOOTING WIDTH b IN.	SHEAR STRENGTH r_f KIPS/SQ FT
———□	9	8	1.7
— — —○	10	6	1.4
—·—·—△	11	4.5	1.0

* CONTINUOUS REACTION-DISPLACEMENT CURVE FROM ONE STATIC TEST.

** CONSTRUCTED REACTION-DISPLACEMENT CURVE USING R_{MAX} AND z_{MAX} FROM FIVE DYNAMIC TESTS.

Figure 3.2 Dimensionless footing reaction-displacement relation for surface footings.

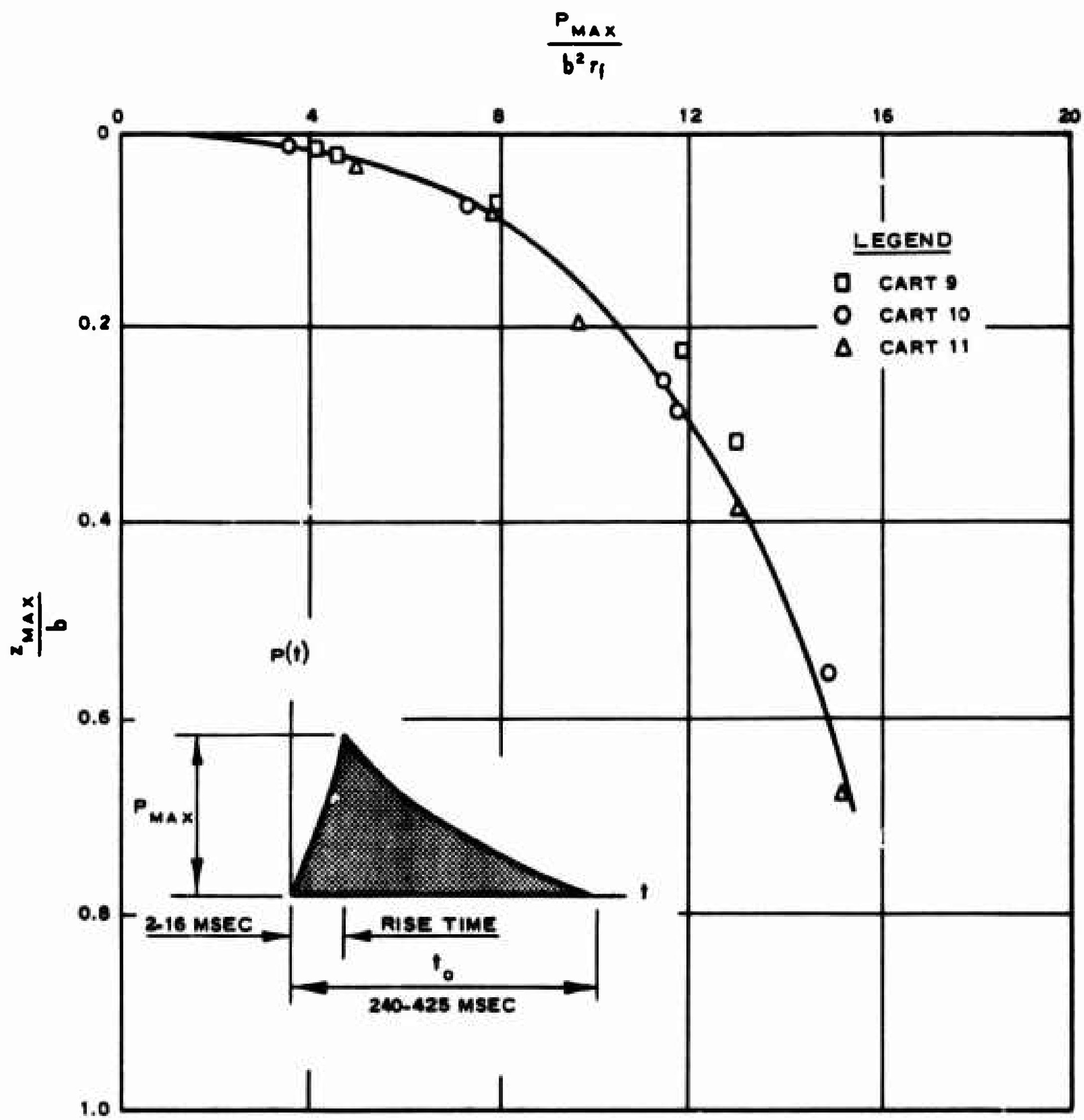


Figure 3.3 Dimensionless relation of maximum applied dynamic column load versus maximum footing displacement for surface footings (Reference 3).

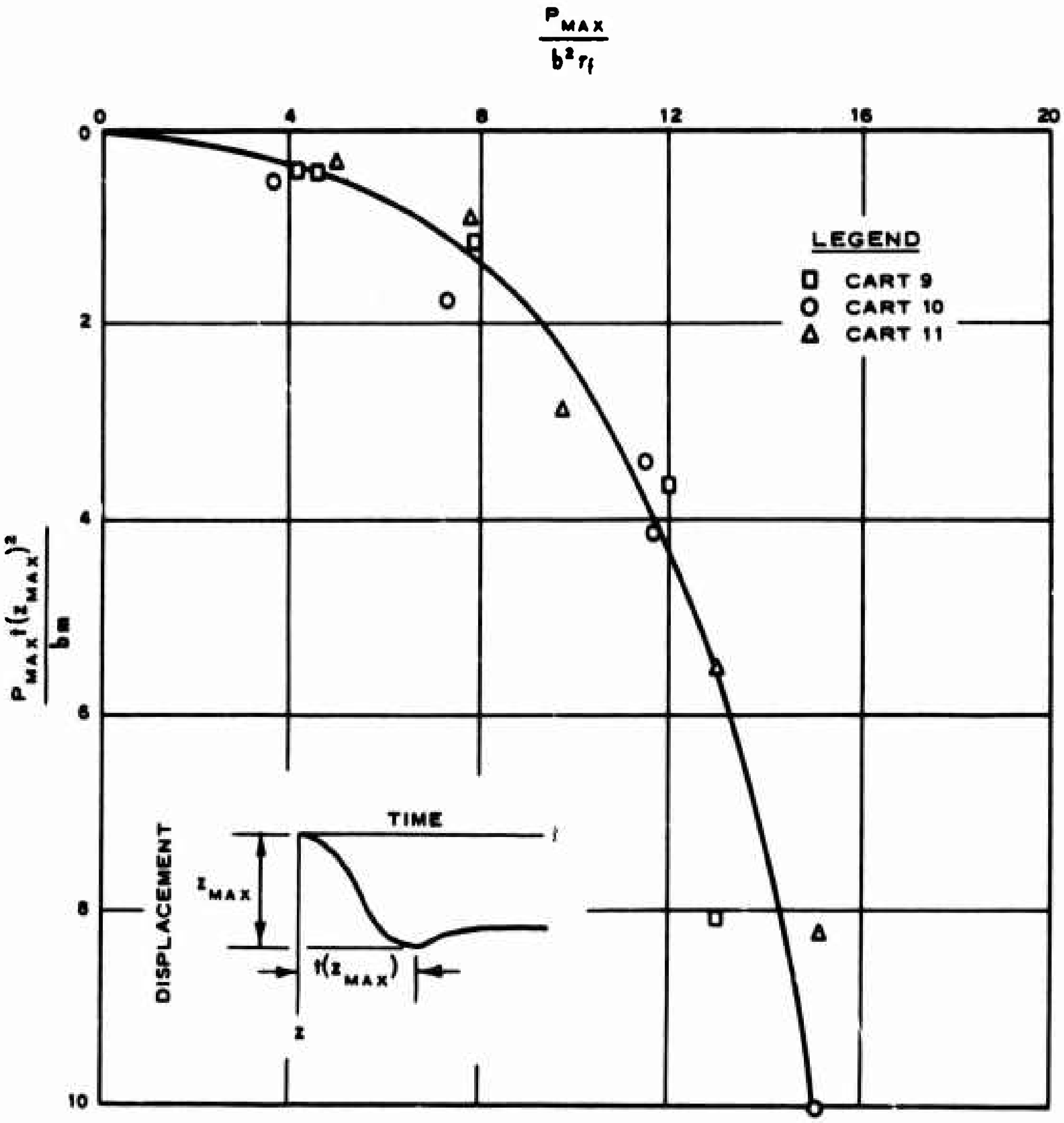
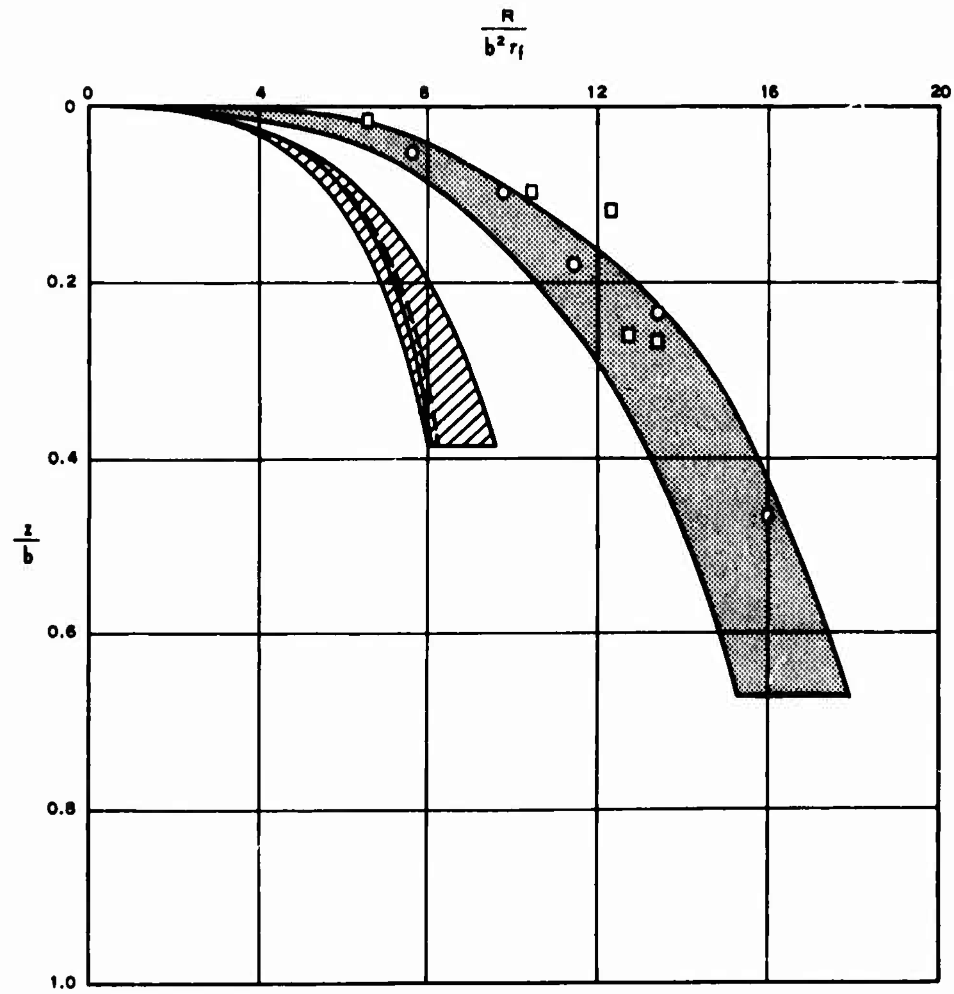


Figure 3.4 Dimensionless relation between maximum applied dynamic column load and time to maximum displacement for surface footings (Reference 3).



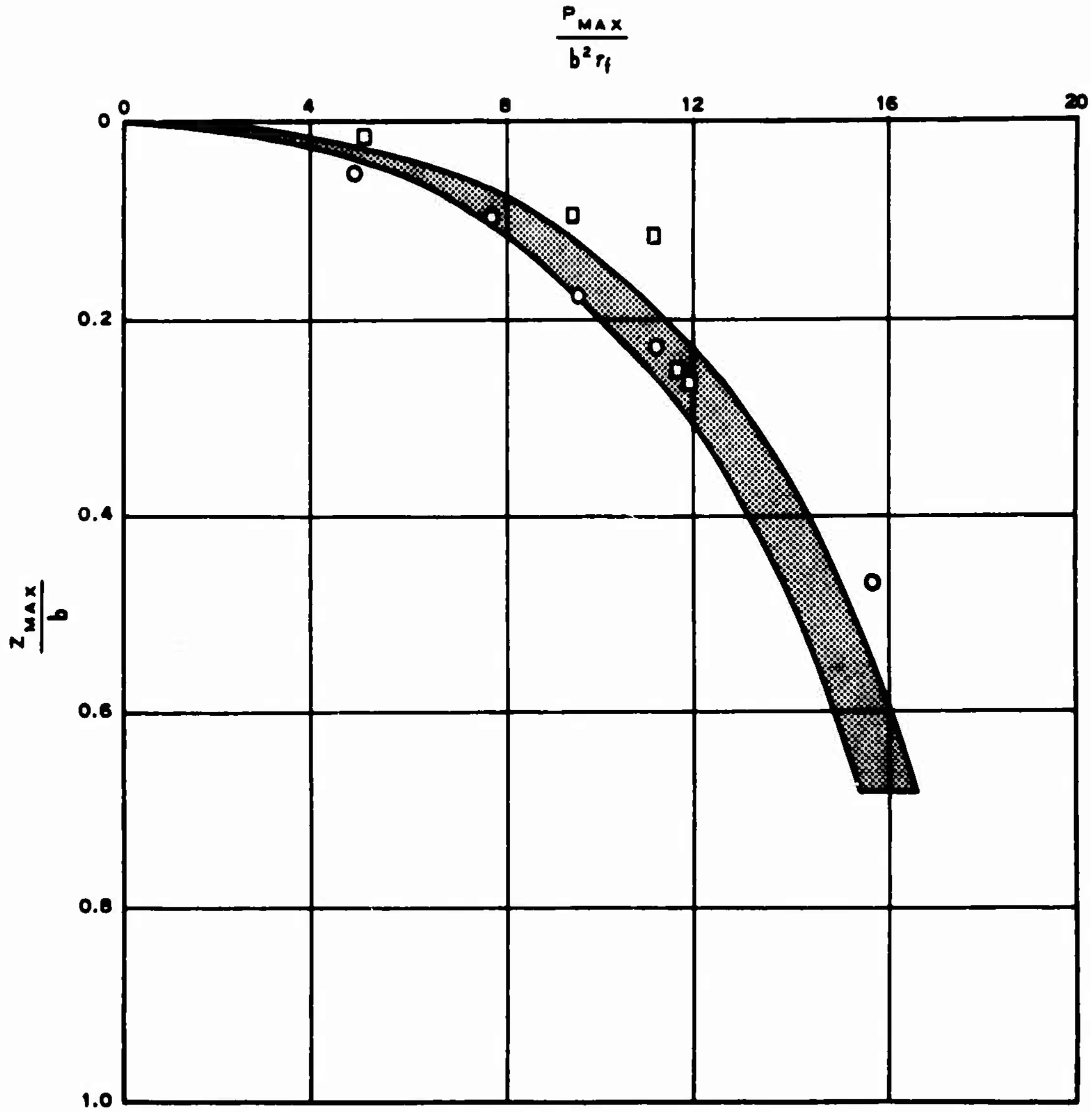
LEGEND

SYMBOL			DEPTH OF BURIAL
STATIC TEST*	DYNAMIC TEST**	CART	PLATE WIDTH D/b
———	□	CART 58	1
— — —	○	CART 59	1
		RANGE FOR CARTS 9, 10, 11	0

* CONTINUOUS LOAD-DISPLACEMENT CURVE FROM ONE STATIC TEST.

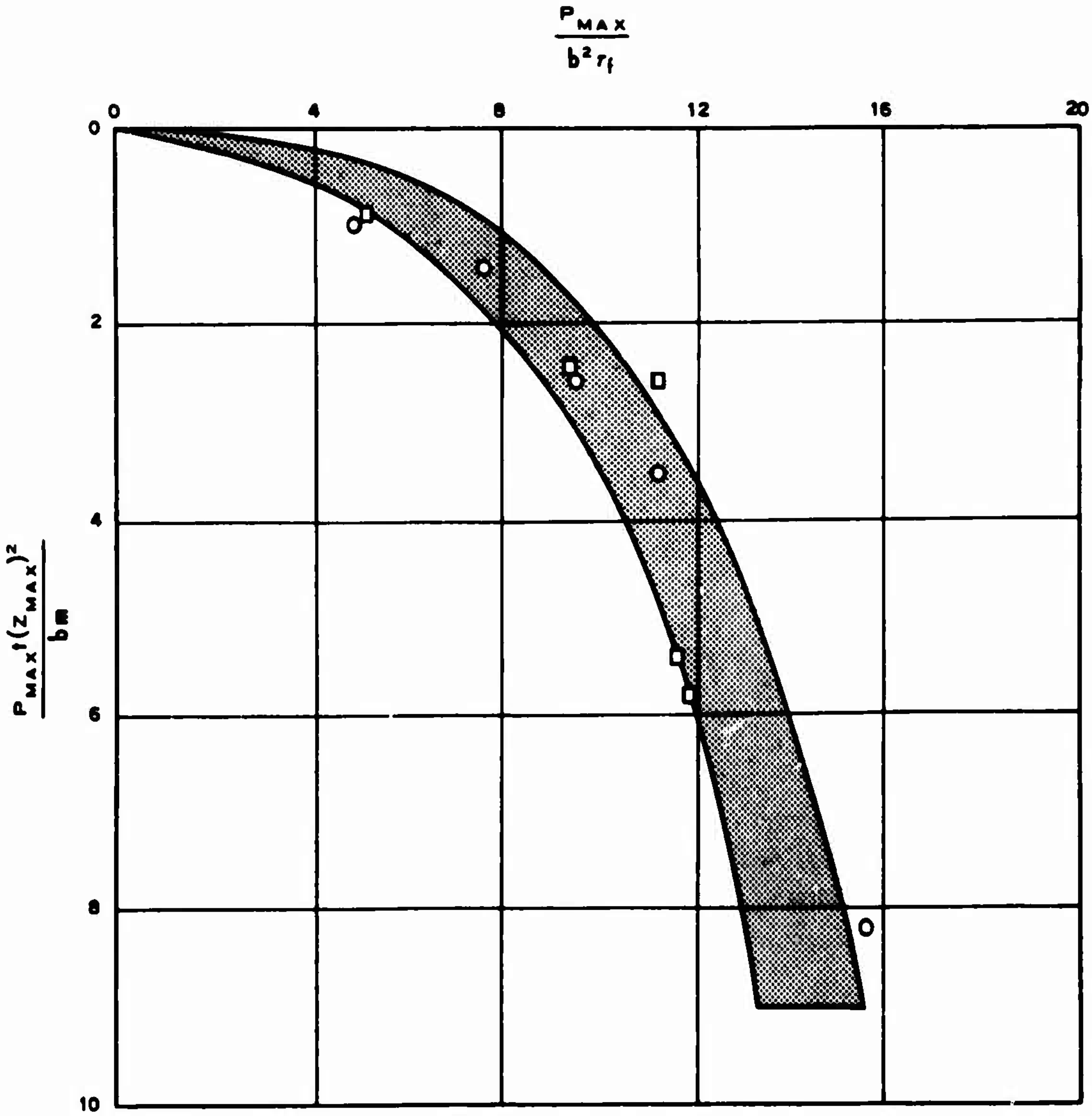
** DYNAMIC TEST POINTS USING VALUES OF R_{MAX} AND Z_{MAX} FROM DYNAMIC TESTS.

Figure 3.5 Effect of shallow depth of burial on resistance parameter versus displacement parameter.



SYMBOL	CART	DEPTH OF BURIAL / PLATE WIDTH D/b
□	CART 58	1
○	CART 59	1
(shaded band)	RANGE FOR CARTS 9, 10, 11	0

Figure 3.6 Effect of shallow depth of burial on strength parameter versus displacement parameter.



LEGEND

SYMBOL	CART	DEPTH OF BURIAL / PLATE WIDTH D/b
□	CART 58	1
O	CART 59	1
(shaded band)	RANGE FOR CARTS 9, 10, 11	0

Figure 3.7 Effect of shallow depth of burial on strength parameter versus inertia parameter.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

The following conclusions are drawn from the results of this investigation:

1. The form of the nondimensional relations developed previously for dynamically loaded surface footings, as given in Figures 3.2, 3.3, and 3.4, also holds for footings at shallow depth of burial in clay.

2. The effect of shallow depth of burial on the nondimensional relations, as shown in Figures 3.5, 3.6, and 3.7, is small and can be considered negligible for design purposes.

4.2 RECOMMENDATIONS

As a result of this investigation it is recommended that the techniques employed in this investigation be used to study the influence of greater depths of burial (up to $D/b = 4$, if necessary) on the response of footings in clay under dynamic loading.

APPENDIX A

SOIL SPECIMEN CONSTRUCTION

A.1 SOIL PROPERTIES

The soil used in this study was a highly plastic clay found as a backswamp deposit alongside the Mississippi River near Vicksburg, Mississippi. It is locally referred to as buckshot clay and is classified as CH in the Unified Soil Classification System.

The clay was air-dried from its natural water content of 30 to 40 percent to about 10 percent, mechanically processed, and stored for later use. The average Atterberg limits for the processed clay were: liquid limit, 60; plastic limit, 23; and plasticity index, 37. The specific gravity of the clay was 2.70. Using the Standard Proctor test, the optimum water content was 23.5 percent and the maximum dry unit weight was 96.2 pcf. Approximate amounts of minerals in the clay, as determined by X-ray diffraction, are as follows:

Clay Fraction 0.2 to 2.0 microns		Clay Fraction less than 0.2 micron	
Montmorillonite	30%	Montmorillonite	60%
Illite	25%	Illite	15%
Kaolinite	25%	Kaolinite	15%
Quartz	20%	K-Feldspar	10%

A.2 SPECIMEN CONSTRUCTION

The procedures used to prepare soil specimens will only be generally described here since a detailed description of the procedures used is presented in Reference 3.

To prepare a soil specimen, the air-dried clay was mixed with water in a pug mill and placed in the soil cart in layers of 3-inch compacted thickness using a mechanical backfill tamper.

The lift at the depth where the footings were to be placed was intentionally compacted 0.50 to 0.75 inch higher than the planned depth of burial. A straightedge was then used to trim away excess soil before the footing was set into place at the proper elevation. The surface was scarified prior to placing the next lift (see Figure A.1).

Openings were made in the specimen directly above the center of each footing to receive the load cell bearing tip, as shown in Figure A.2, and at each of three corners to receive the extension rods of the linear-motion potentiometers used to measure footing displacement (see Figure A.3).

The top surface of the cart was sealed with a polyethylene membrane and the clay allowed to cure for approximately two days to allow for equalization of moisture.

Figure A.1 Footings positioned at proper elevation and surface scarified.

Figure A.2 Cutting opening in soil specimen for load cell bearing tip.

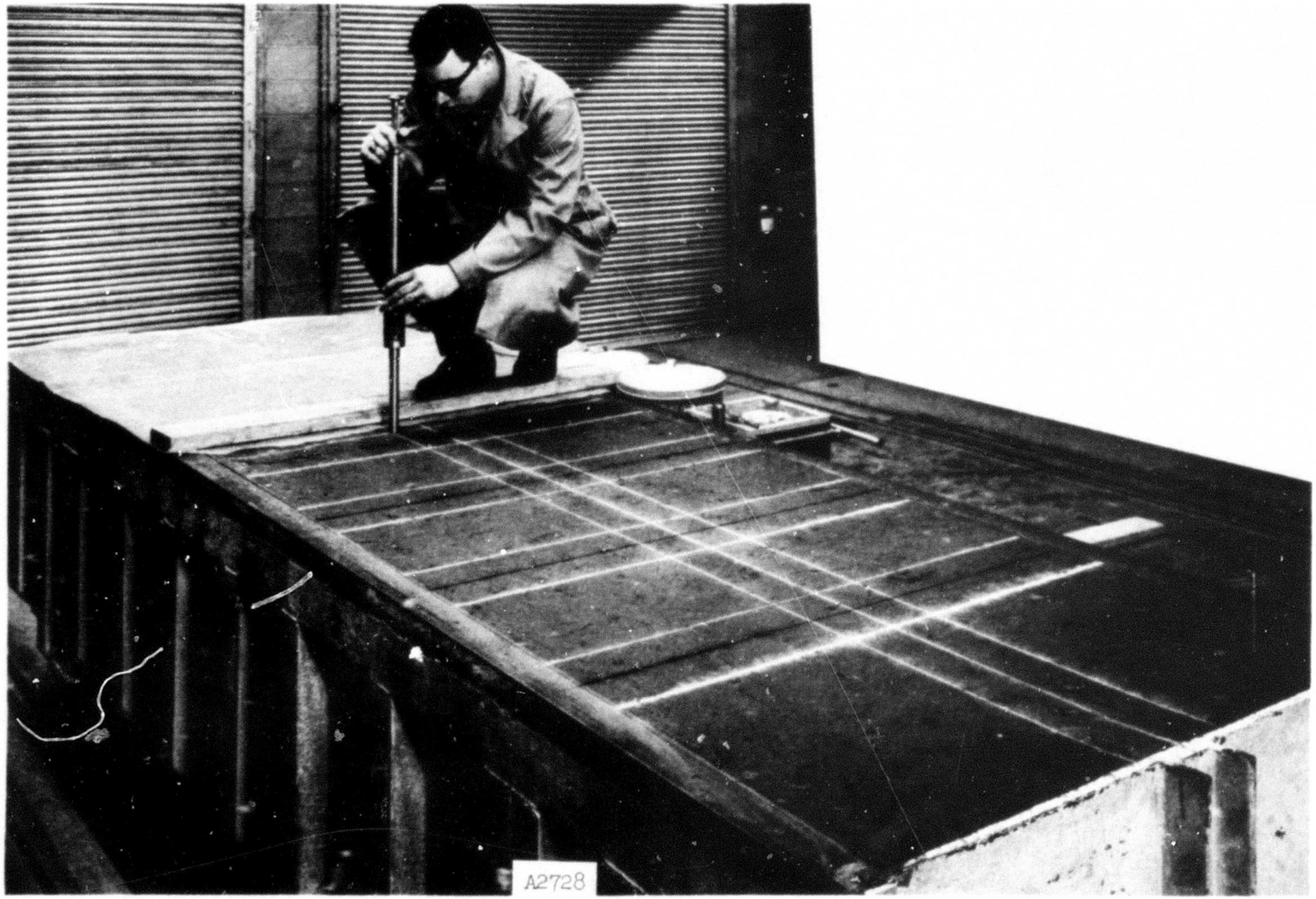


Figure A.3 Cutting opening in soil specimen for extension rod of potentiometer.

APPENDIX B

TEST METHODS AND PROCEDURES

B.1 TEST SYSTEM

The test system used in this study consisted of a dynamic loading machine capable of producing controlled dynamic and static loads of up to 50,000 pounds; square aluminum plate footings; a compacted specimen of highly plastic, nearly saturated clay in a movable track-mounted soil cart; and electronic instrumentation support. A typical test setup is shown in Figure B.1. Detailed descriptions of the dynamic loading machine and its operation are presented in References 1 and 3.

B.2 MECHANICAL VANE SHEAR TEST

Immediately before each footing test, mechanical vane shear tests were conducted using the device shown in Figures B.2 and B.3 to provide an estimate of the soil strength for the proposed footing test.

Preliminary tests using the mechanical vane shear device in a 3- by 3- by 1-foot-deep steel box filled with soil, compacted with the same procedure used in the soil cart specimens, indicated the need for control over the frictional component of vane resistance when testing at different depths. The mechanical vane was modified by constructing a projection on the vane shaft, as shown in Figure B.2,

that produced constant friction between the vane shaft and the clay when operating the vane at any depth below 2 inches. The preliminary tests also showed that a vane resistance versus depth profile existed within each 3-inch-thick compacted lift.

For each footing test, mechanical vane tests were conducted in a 6- by 8-inch grid pattern within a 12- by 16-inch area around each footing location and readouts of torque versus angle of rotation were obtained. The depths at which vane tests were conducted were chosen so that each depth would represent readings of either the top, middle, or bottom of a lift both above and below the footing. The mechanical vane shear device was calibrated before and after each series of tests using a moment arm of known length and weights.

B.3 DYNAMIC AND STATIC TEST PROCEDURES

Detailed descriptions of the dynamic and static test procedures are given in Reference 3. The data from a typical dynamic footing test are shown in Figure B.4. An idealized curve has been fitted to the computed column load curve and used for analysis purposes. The various time parameters determined from the idealized curve are the rise time (t_r), hold time (t_h), decay time (t_d), and positive-phase duration (t_o), as defined in the figure.

B.4 SOIL SAMPLING AND TESTING

After all dynamic and static footing tests on a soil cart were

completed, ten samples were taken from the top 3 inches of the specimen for water content and wet unit weight determinations. Following this, six 8-inch-diameter by 9-inch-high undisturbed block samples were taken at the elevation of the footing tests.

Three strain-controlled, unconfined compression tests were conducted on 1.4-inch-diameter by 3.0-inch-high specimens from the top 4 inches of each of the six block samples. Strain-controlled, unconsolidated-undrained, triaxial tests, with confining pressures of 1, 3, and 6 kips per square foot, were conducted on 1.4-inch-diameter by 3-inch-high specimens taken from the top 4 inches of two block samples from each soil cart. The triaxial test specimens were taken from the two block samples for which the unconfined compression test results were nearest the average for the cart. All strength tests followed standard Corps of Engineers procedures (Reference 14).

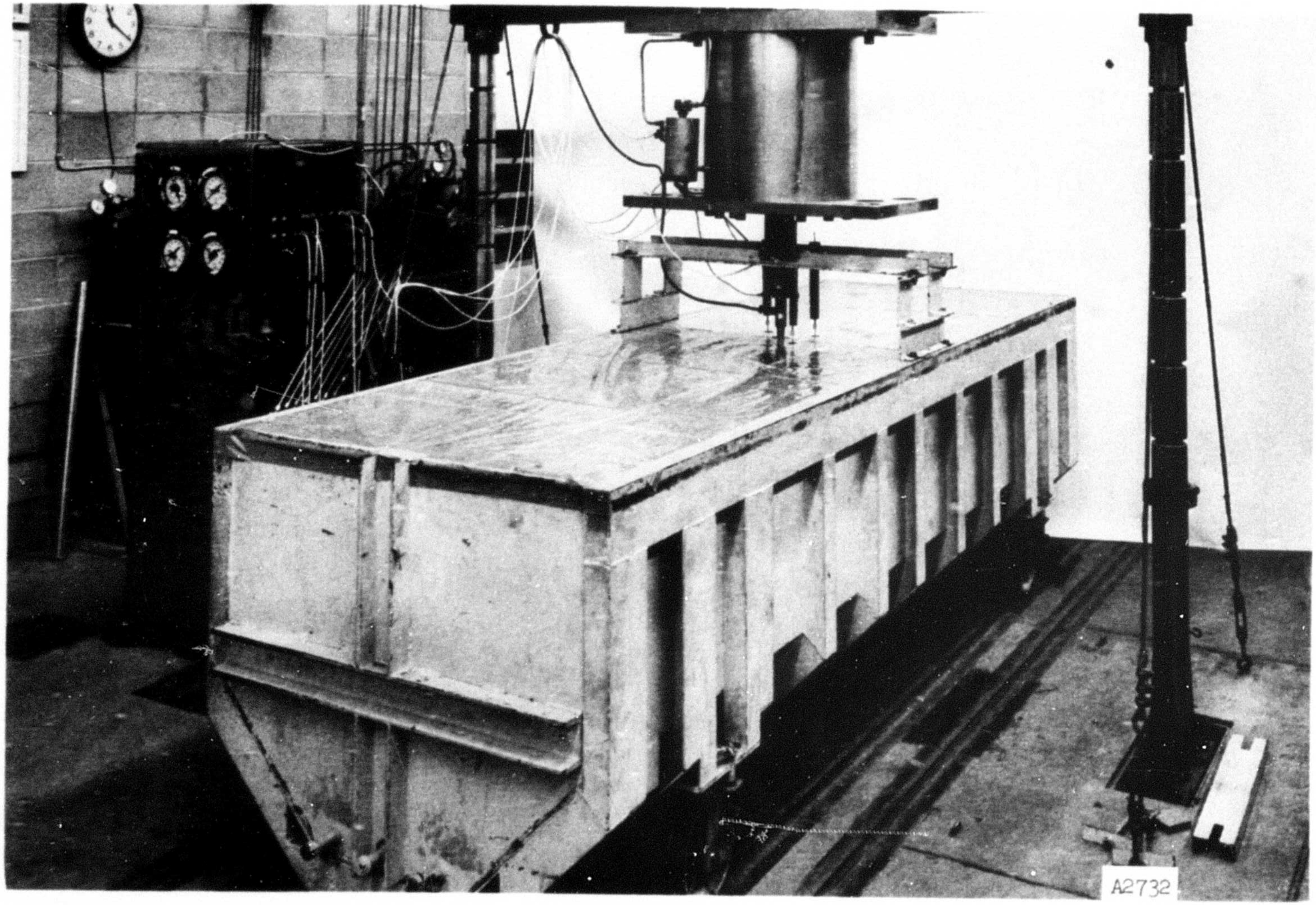


Figure B.1 Dynamic loading machine and soil specimen set up for dynamic test.

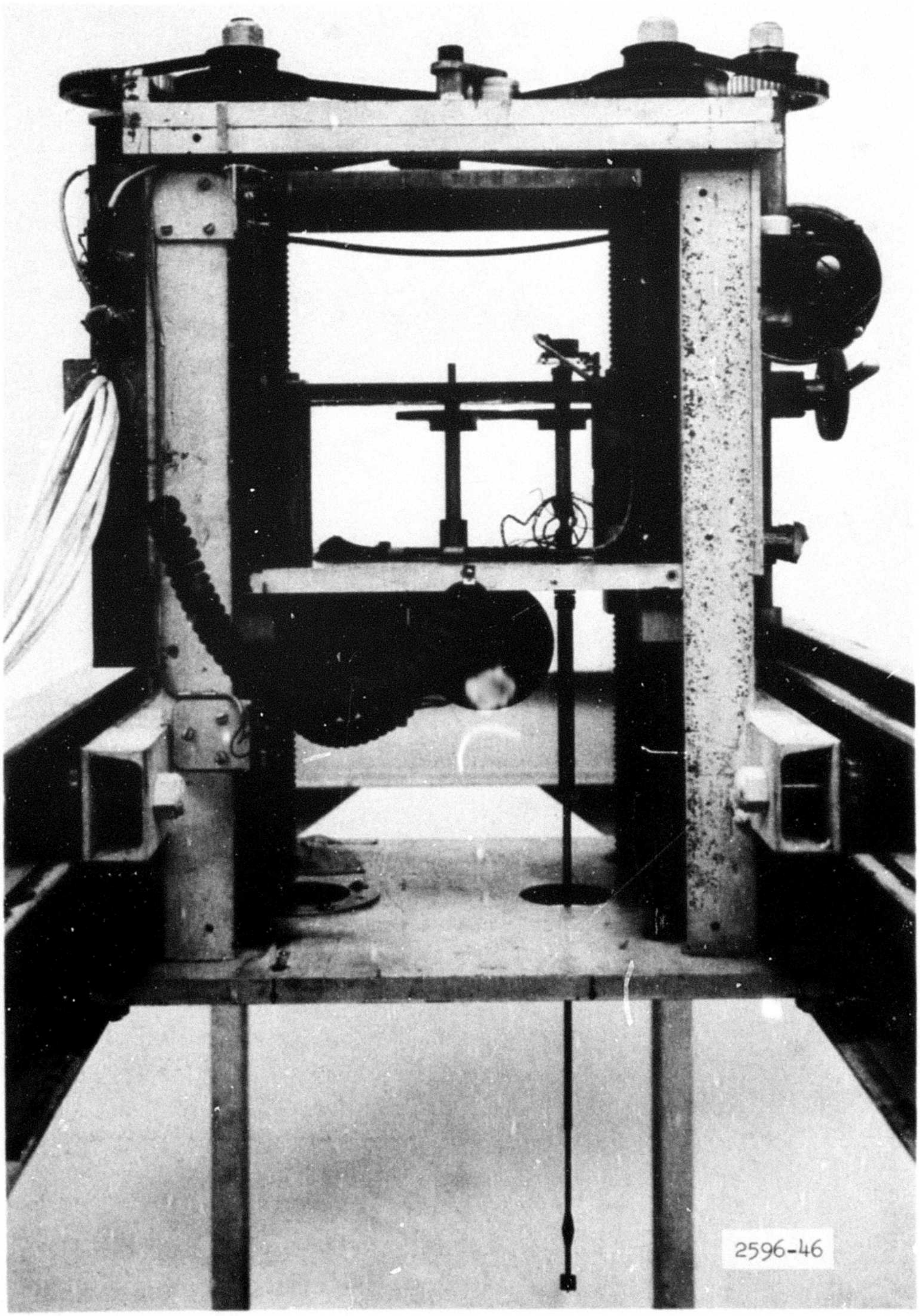


Figure B.2 Components of mechanical vane shear device.

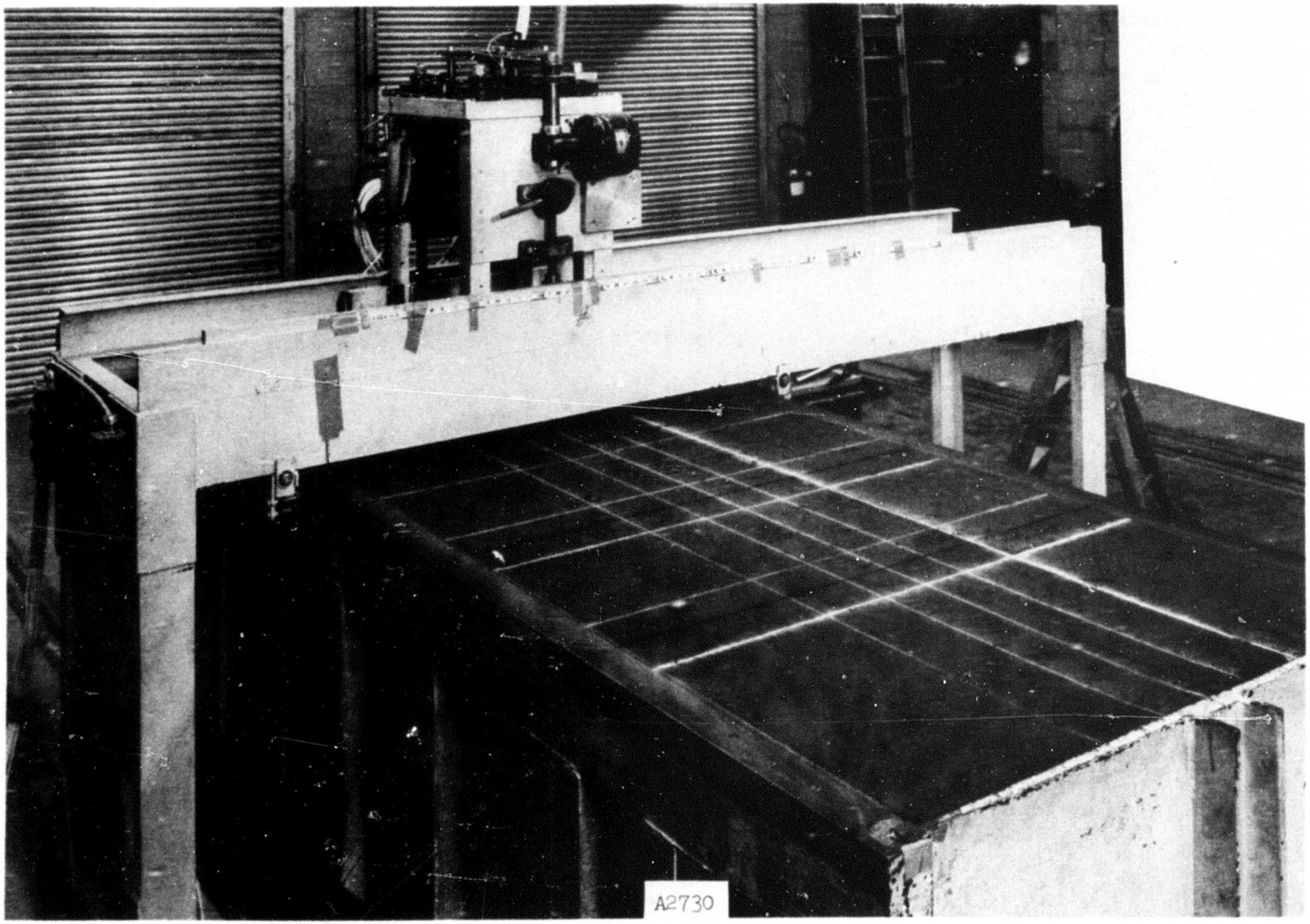


Figure B.3 Mechanical vane shear device in operation.

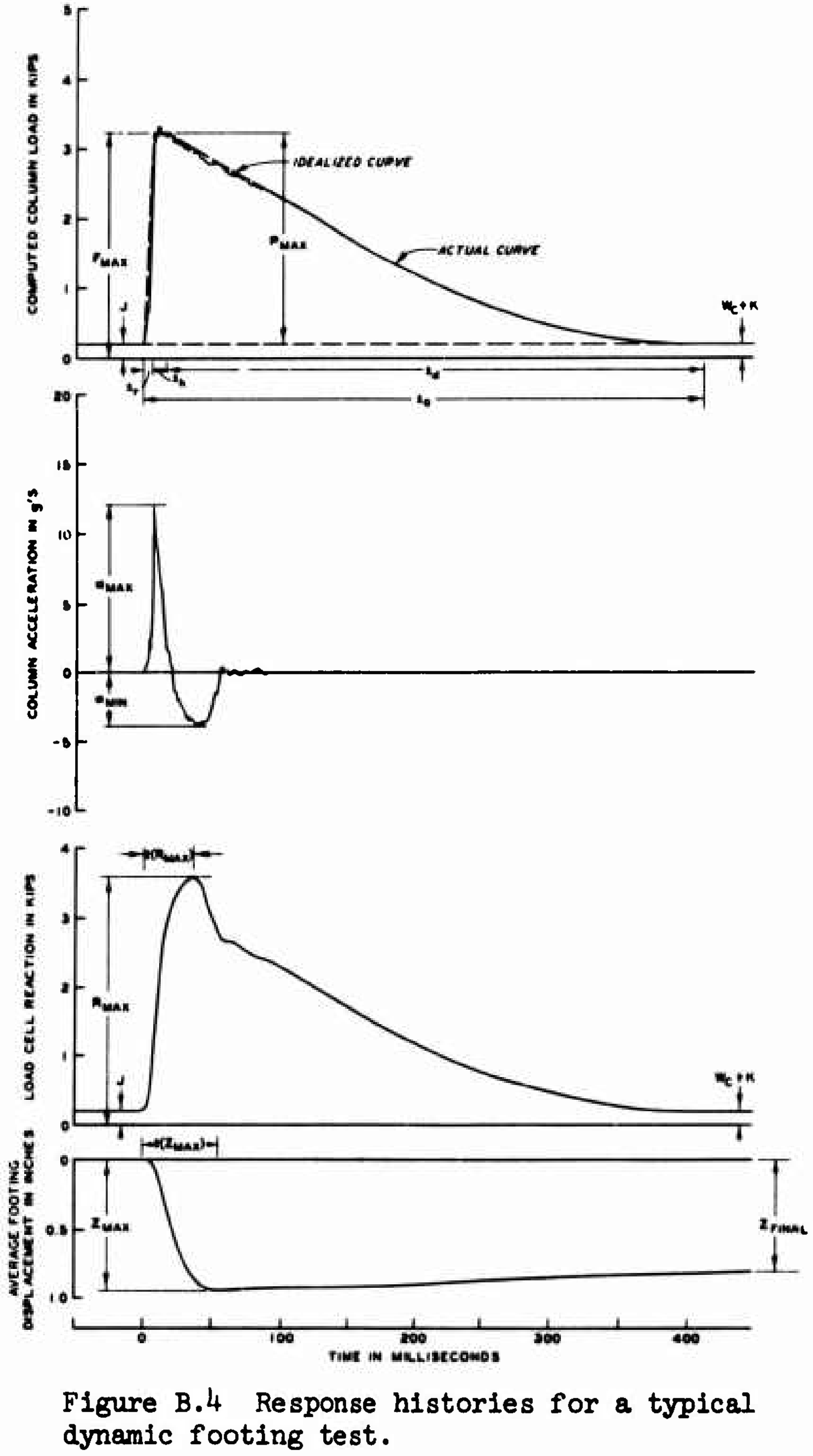


Figure B.4 Response histories for a typical dynamic footing test.

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APPENDIX C

SOIL TEST RESULTS

TABLE C.1 VANE SHEAR RESISTANCE AND UNIT WEIGHT TESTS, PART 58

(A) Construction Control Tests

Compaction-Layer Depth	In-Place Unit Wt: Station	Offset	Water Content	Wet Unit Wt	Dry Unit Wt	Hand-Operated Vane Resistance: Average	Range
in.	ft	in.	percent	pcf	pcf	kips/sq ft	kips/sq ft
0-2.25	1+25	7	27.3	120.0	94.3	2.59	2.27-3.09
	5+75	33	27.3	119.3	93.7		
	10+75	7	27.2	118.9	93.5		
2.25-4.75	2+75	33	26.4	119.6	94.6	2.70	1.99-3.29
	4+25	7	26.9	118.9	93.7		
	7+25	7	27.0	119.2	93.9		
	9+00	33	27.5	119.3	93.6		
4.75-7.75	1+25	7	26.4	120.1	95.0	2.57	2.10-3.05
	5+75	33	27.3	118.3	92.9		
	10+75	7	27.0	119.3	93.9		
7.75-10.75	2+75	33	27.5	119.2	93.5	2.36	1.98-2.85
	4+25	7	26.9	118.9	93.7		
	7+25	7	27.7	119.1	93.3		
	9+00	33	27.5	119.3	93.6		
10.75-13.75	1+25	7	27.3	119.2	93.6	2.51	2.14-2.92
	5+75	33	27.0	119.0	93.7		
	10+75	7	27.7	118.8	93.0		
13.75-16.75	2+75	33	25.6	120.1	95.6	2.50	2.06-3.04
	4+25	7	26.5	121.2	95.8		
	7+25	7	26.7	120.1	94.8		
	9+00	33	26.8	119.6	94.3		
Avg			27.0	119.4	94.0	2.54	

(B) Pretest Mechanical Vane Tests

Test No.	Plate Location: Station	Offset	Average Vane Resistance
	ft	in.	kips/sq ft
58-1	2+00	20	3.48
58-2	3+50	20	2.83
58-3	5+00	20	3.34
58-4	6+50	20	2.70
58-5	8+00	20	3.08
58-6	10+00	20	3.51
Avg			3.16

(C) Posttest Unit Wt Test

Station	Offset	Water Content	Wet Unit Wt	Dry Unit Wt
ft	in.	percent	pcf	pcf
0+50	20	26.6	120.3	95.0
3+50	7	26.4	120.6	95.4
3+50	33	26.7	120.9	95.4
5+00	7	27.0	119.3	93.9
5+00	33	27.0	120.4	94.8
6+50	7	26.9	120.3	94.8
6+50	33	26.9	118.7	93.5
8+00	7	27.5	119.8	94.0
8+00	33	26.8	119.2	94.0
11+50	20	27.3	118.9	93.4
Avg		26.9	119.8	94.4

TABLE C.2 VANE SHEAR RESISTANCE AND UNIT WEIGHT TESTS, CART 59

(A) Construction Control Tests

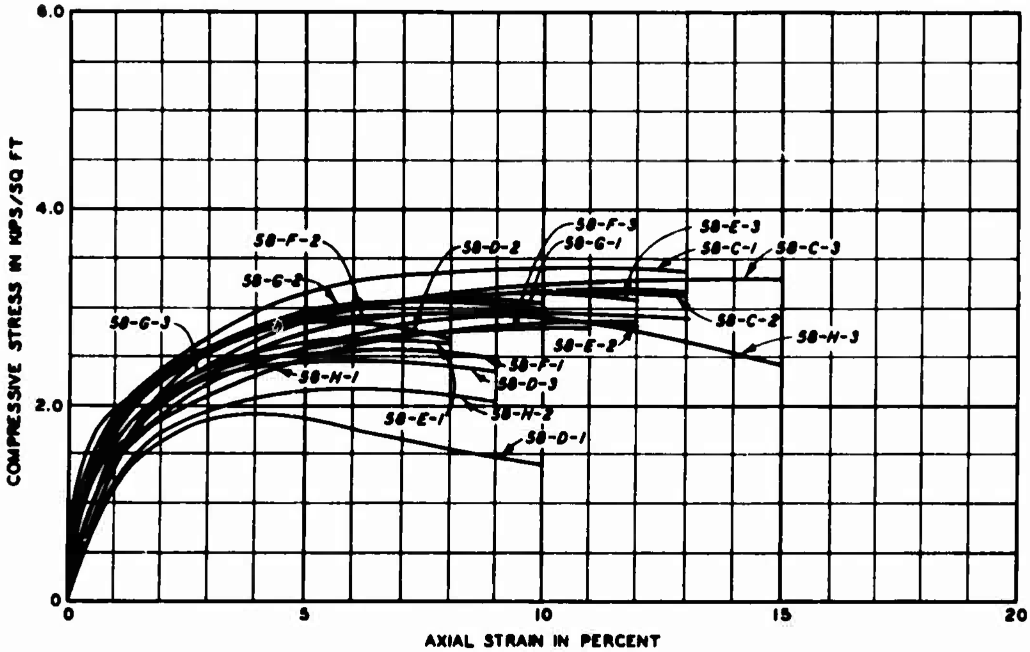
Compaction-Layer Depth	In-Place Unit Wt					Hand-Operated Vane Resistance	
	Station	Offset	Water Content	Wet Unit Wt	Dry Unit Wt	Average	Range
in.	ft	in.	percent	pcf	pcf	kips/sq ft	kips/sq ft
0-1.75	2+75	33	28.7	117.7	91.5	1.59	1.24-1.80
	4+25	7	29.6	117.1	90.4		
	7+25	7	29.6	117.3	90.5		
1.75-4.0	1+00	7	30.7	116.4	89.1	1.66	1.52-2.06
	5+75	33	29.1	118.0	91.4		
	10+25	7	28.6	119.1	92.6		
4.0-7.0	1+00	7	30.5	116.9	89.6	1.57	1.39-1.76
	5+75	33	29.2	117.8	91.2		
	10+25	7	28.7	116.7	90.7		
7.0-10.25	2+75	33	29.4	116.3	89.9	1.48	0.97-1.84
	4+25	7	28.3	117.8	91.8		
	7+25	7	28.7	116.5	90.5		
	11+00	33	29.0	116.6	90.4		
10.25-13.5	1+00	7	29.0	116.6	90.4	1.51	1.31-2.06
	5+75	33	28.8	116.1	90.1		
	10+25	7	28.5	117.8	91.7		
13.5-16.75	2+75	33	27.6	117.3	91.9	1.77	1.70-2.02
	4+25	7	28.2	117.2	91.4		
	7+25	7	28.4	117.4	91.4		
	11+00	33	28.7	117.9	91.6		
Avg			29.0	117.2	90.9	1.60	

(B) Pretest Mechanical Vane Tests

Test No.	Plate Location		Average Vane Resistance
	Station	Offset	
	ft	in.	kips/sq ft
59-1	2+00	20	2.26
59-2	3+50	20	2.30
59-3	5+00	20	2.26
59-4	6+50	20	2.16
59-5	8+00	20	2.28
59-6	10+00	20	2.42
Avg			2.28

(C) Posttest Unit Wt Tests

Station	Offset	Water Content	Wet Unit Wt	Dry Unit Wt
ft	in.	percent	pcf	pcf
1+75	33	28.8	117.8	91.5
3+50	7	28.3	118.8	92.6
5+00	7	28.5	117.0	91.1
5+00	33	28.8	117.7	91.4
6+50	7	28.7	118.3	91.9
6+50	33	28.7	119.4	92.8
8+25	7	29.2	117.7	91.1
8+25	33	29.0	117.3	90.9
10+25	33	29.1	117.9	91.3
11+00	20	30.3	116.4	89.3
Avg		28.9	117.8	91.4

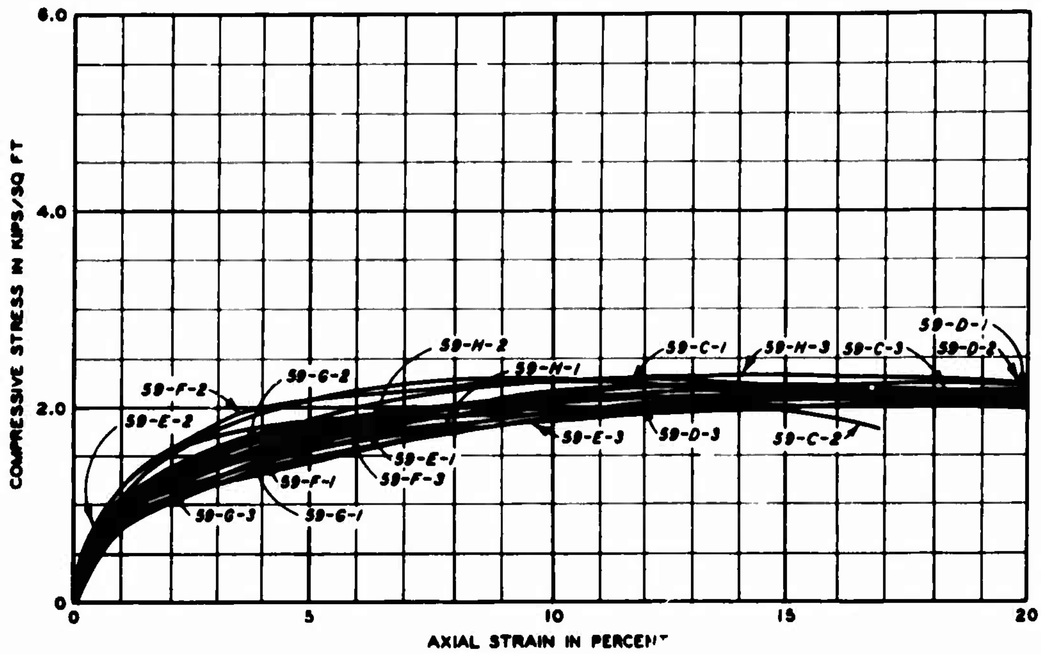


BLOCK				TEST*	INITIAL CONDITIONS				UNCONF COMP STRESS	SHEAR STRESS	TIME** TO FAILURE	FAILURE STRAIN
NUMBER	LOCATION				WATER CONTENT	DRY UNIT WT	VOID RATIO	SAT.				
	STATION FT	OFFSET IN.	DEPTH IN.		%	LB PER CU FT		%	KIPS PER SQ FT	KIPS PER SQ FT	MIN	%
58	2+75	7	8-17	C-1	26.8	95.0	0.761	94.4	3.40	1.70	16	10.0
				C-2	27.2	93.1	0.797	91.5	3.16	1.58	16	10.0
				C-3	27.0	93.1	0.797	90.8	3.28	1.64	25	15.0
58	2+50	33	8-17	D-1	27.0	88.7	0.865	83.6	1.92	0.96	7	4.5
				D-2	26.3	90.9	0.841	83.8	2.84	1.42	13	6.0
				D-3	27.2	91.9	0.820	88.9	2.46	1.23	15	5.5
58	5+75	7	8-17	E-1	26.9	91.2	0.834	86.4	2.66	1.33	15	6.0
				E-2	27.3	93.4	0.791	92.5	2.94	1.47	21	9.0
				E-3	27.1	92.9	0.800	90.8	3.12	1.56	22	9.0
58	7+25	33	8-17	F-1	27.3	92.0	0.818	89.4	2.56	1.28	17	7.0
				F-2	27.5	93.5	0.788	93.4	2.94	1.47	24	10.0
				F-3	27.5	93.1	0.797	92.5	2.80	1.40	32	11.0
58	9+00	7	8-17	G-1	27.1	94.6	0.780	94.4	3.06	1.53	11	8.0
				G-2	27.1	93.8	0.784	92.6	3.06	1.53	14	8.0
				G-3	27.1	93.5	0.788	92.1	3.16	1.58	22	11.0
58	1+00	33	8-17	H-1	28.0	93.6	0.780	95.2	2.86	1.43	18	12.0
				H-2	27.3	92.3	0.813	90.0	2.18	1.09	9	5.5
				H-3	26.9	92.8	0.803	88.8	2.90	1.45	16	10.0
				AVERAGE	27.1	92.8	0.803	90.7	2.85	1.43	17	8.8

* STRAIN-CONTROLLED TEST ON 1.4-IN.-DIAM × 3.01-IN.-HIGH SPECIMEN TAKEN FROM 0- TO 4-IN. DEPTH OF THE BLOCK SAMPLE.

** RATE OF STRAIN CHANGED FROM 1% PER MINUTE TO 2.5% PER MINUTE AT FROM 4 TO 8% AXIAL STRAIN.

Figure C.1 Unconfined compression test results, cart 58.

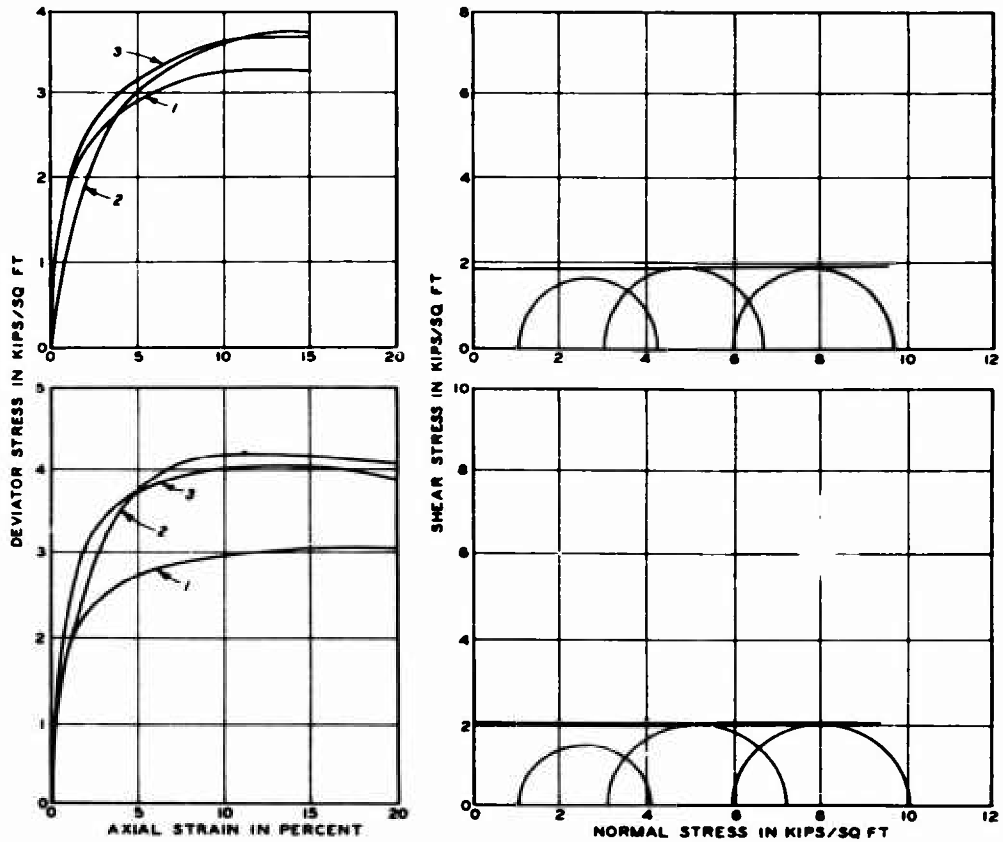


BLOCK				TEST*	INITIAL CONDITIONS				UNCONF COMP STRESS	SHEAR STRESS	TIME** TO FAILURE	FAILURE STRAIN
NUMBER	LOCATION				WATER CONTENT	DRY UNIT WT LB PER CU FT	VOID RATIO	SAT.				
	STATION FT	OFFSET IN.	DEPTH IN.		%			%	KIPS PER SQ FT	KIPS PER SQ FT	MIN	%
59	2+75	7	4-13.5	C-1	28.2	91.4	0.830	91.0	2.28	1.14	14	9.3
				C-2	28.6	90.0	0.858	89.2	2.00	1.00	15	11.0
				C-3	28.4	92.0	0.818	93.0	2.18	1.09	18	15.0
59	4+25	33	4-13.5	D-1	28.5	91.0	0.838	91.1	2.12	1.06	23	15.0
				D-2	28.2	90.1	0.857	88.2	2.22	1.11	21	15.0
				D-3	28.7	91.0	0.838	91.8	2.10	1.05	20	13.2
59	5+75	7	4-13.5	E-1	28.4	90.8	0.843	90.3	2.16	1.08	20	15.0
				E-2	28.8	90.8	0.843	91.6	2.18	1.09	18	13.5
				E-3	28.8	91.7	0.824	93.7	2.04	1.02	24	17.0
59	7+25	33	4-13.5	F-1	29.3	90.0	0.858	91.4	1.98	0.99	21	14.5
				F-2	28.5	91.4	0.830	92.0	2.26	1.13	15	12.2
				F-3	28.7	90.2	0.855	90.0	2.16	1.08	19	14.5
59	9+00	7	4-13.5	G-1	29.0	90.1	0.857	90.7	2.10	1.05	19	13.9
				G-2	29.0	90.8	0.843	92.2	2.16	1.08	25	13.5
				G-3	28.9	90.7	0.845	91.6	2.08	1.04	22	18.0
59	9+00	33	4-13.5	H-1	28.9	90.0	0.858	89.8	2.06	1.03	22	17.5
				H-2	28.7	90.8	0.843	91.2	2.12	1.06	18	14.5
				H-3	28.5	92.0	0.818	93.4	2.30	1.15	18	13.0
				AVERAGE	28.7	90.8	0.842	91.2	2.14	1.07	20	14.2

* STRAIN-CONTROLLED TEST ON 1.4-IN.-DIAM × 3.01-IN.-HIGH SPECIMEN TAKEN FROM 0- TO 4-IN. DEPTH OF THE BLOCK SAMPLE.

** RATE OF STRAIN CHANGED FROM 1% PER MINUTE TO 2.5% PER MINUTE AT FROM 4 TO 8% AXIAL STRAIN.

Figure C.2 Unconfined compression test results, cart 59.

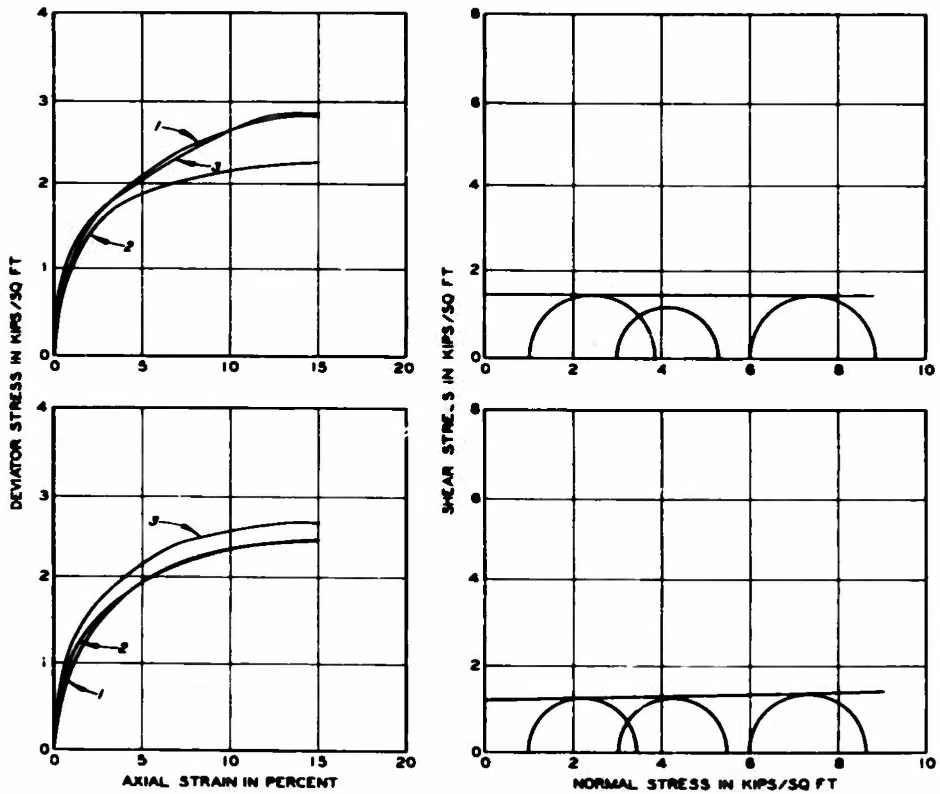


BLOCK	58-G		
STATION	9+00		
OFFSET, IN.	7		
DEPTH, IN.	0- 7		
SAMPLE*	1	2	3
WATER CONTENT, %	27.2	27.3	27.4
DRY DENSITY, LB/CU FT	92.8	93.8	92.8
VOID RATIO	0.803	0.784	0.880
SATURATION, %	90.8	93.3	83.4
MAXIMUM DEVIATOR STRESS, KIPS/SQ FT	3.24	3.66	3.68
MINOR PRINCIPAL STRESS, KIPS/SQ FT	1.00	3.00	6.00
MAJOR PRINCIPAL STRESS, KIPS/SQ FT	4.24	6.66	9.68
TIME TO FAILURE, MIN	18	22	22
FAILURE STRAIN, %	5.5	10.5	10.5
APPARENT COHESION, KIPS/SQ FT	1.80		
APPARENT ANGLE OF SHEARING RESISTANCE, DEGREES	0.5		

BLOCK	58-E		
STATION	5+75		
OFFSET, IN.	7		
DEPTH, IN.	0-17		
SAMPLE*	1	2	3
WATER CONTENT, %	27.1	26.8	27.1
DRY DENSITY, LB/CU FT	92.1	93.6	93.6
VOID RATIO	0.817	0.788	0.788
SATURATION, %	88.9	91.1	92.2
MAXIMUM DEVIATOR STRESS, KIPS/SQ FT	3.04	4.16	4.04
MINOR PRINCIPAL STRESS, KIPS/SQ FT	1.00	3.00	6.00
MAJOR PRINCIPAL STRESS, KIPS/SQ FT	4.04	7.16	10.04
TIME TO FAILURE, MIN	40	27	32
FAILURE STRAIN, %	15	15	12.5
APPARENT COHESION, KIPS/SQ FT	2.12		
APPARENT ANGLE OF SHEARING RESISTANCE, DEGREES	0		

* STRAIN-CONTROLLED TEST ON 1.4-IN.-DIAM BY 3.01-IN.-HIGH SPECIMENS OBTAINED FROM 0- TO 4-IN. DEPTH.

Figure C.3 Unconsolidated-undrained triaxial tests, cart 58.



BLOCK	59-H		
STATION	9+00		
OFFSET, IN.	33		
DEPTH, IN.	4-13.5		
SAMPLE*	1	2	3
WATER CONTENT, %	28.6	29.7	28.8
DRY DENSITY, LB/CU FT	90.6	88.8	89.7
VOID RATIO	0.847	0.884	0.865
SATURATION, %	90.5	90.0	89.2
MAXIMUM DEVIATOR STRESS, KIPS/SQ FT	2.84	2.26	2.84
MINOR PRINCIPAL STRESS, KIPS/SQ FT	1.00	3.00	6.00
MAJOR PRINCIPAL STRESS, KIPS/SQ FT	3.84	5.26	8.84
TIME TO FAILURE, MIN	20	20	20
FAILURE STRAIN, %	15.0	15.0	15.0
APPARENT COHESION, KIPS/SQ FT	1.42		
APPARENT ANGLE OF SHEARING RESISTANCE, DEGREES	0		

BLOCK	59-D		
STATION	4+25		
OFFSET, IN.	33		
DEPTH, IN.	4-13.5		
SAMPLE*	1	2	3
WATER CONTENT, %	28.9	29.3	29.3
DRY DENSITY, LB/CU FT	91.1	89.6	89.9
VOID RATIO	0.836	0.867	0.861
SATURATION, %	92.6	90.6	91.2
MAXIMUM DEVIATOR STRESS, KIPS/SQ FT	2.44	2.46	2.68
MINOR PRINCIPAL STRESS, KIPS/SQ FT	1.00	3.00	6.00
MAJOR PRINCIPAL STRESS, KIPS/SQ FT	3.44	5.46	8.68
TIME TO FAILURE, MIN	20	20	18
FAILURE STRAIN, %	15.0	15.0	15.0
APPARENT COHESION, KIPS/SQ FT	1.16		
APPARENT ANGLE OF SHEARING RESISTANCE, DEGREES	1.5		

* STRAIN-CONTROLLED TEST ON 1.4-IN.-DIAM BY 3.01-IN.-HIGH SPECIMENS OBTAINED FROM 0- TO 4-IN. DEPTH.

Figure C.4 Unconsolidated-undrained triaxial tests, cart 59.

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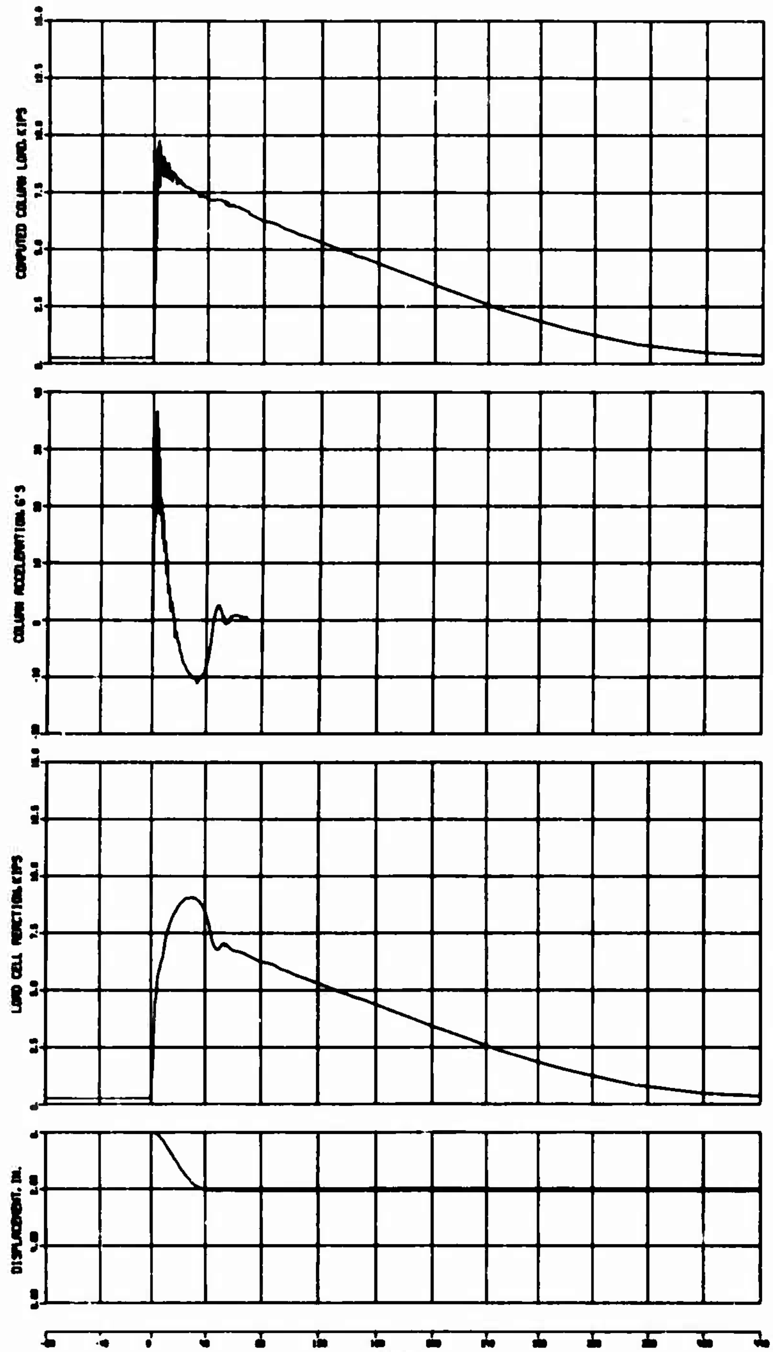
APPENDIX D

DETAILED RESULTS OF FOOTING TESTS



b = 8 IN.
D/b = 1
T_f = 160 KIPS/SQ FT

Figure D.1 Dynamic test 58-1.

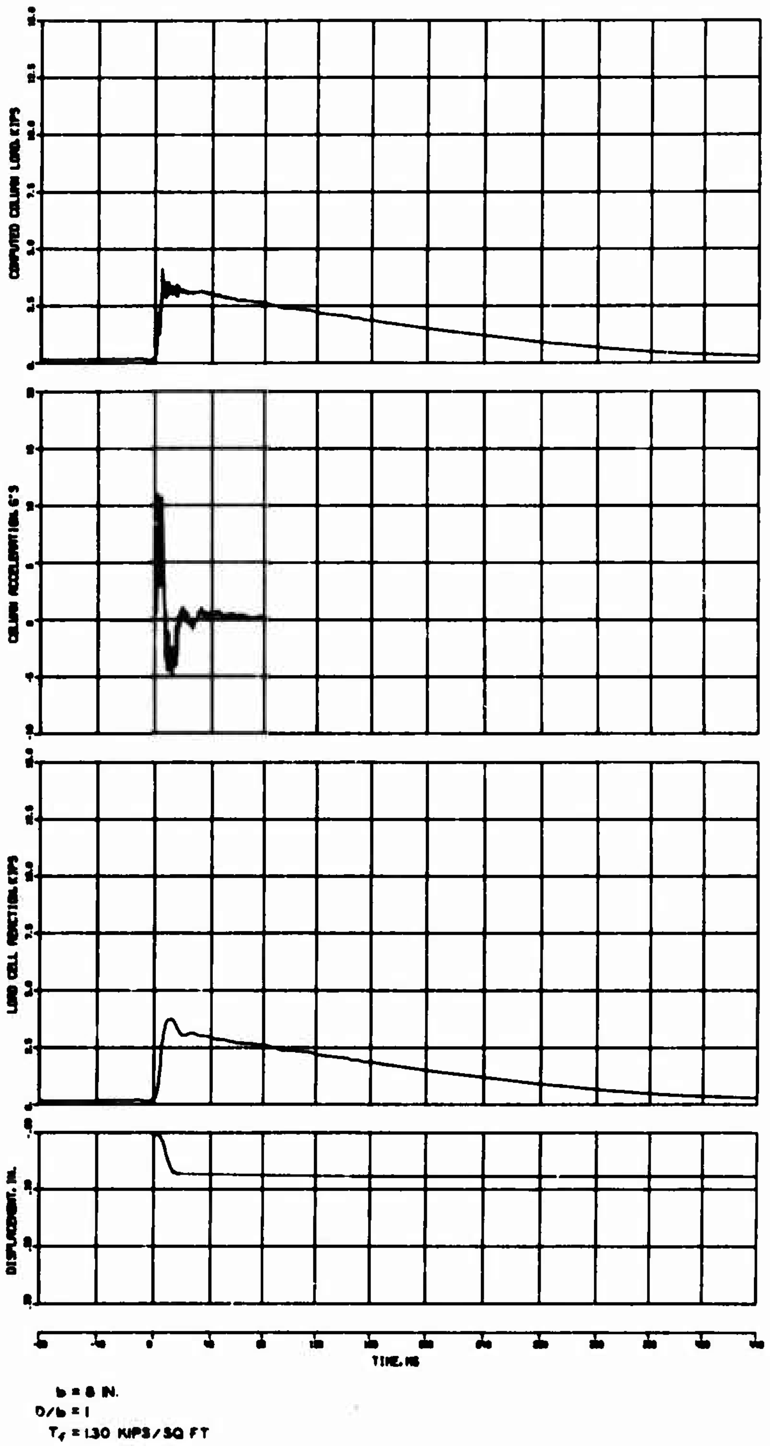
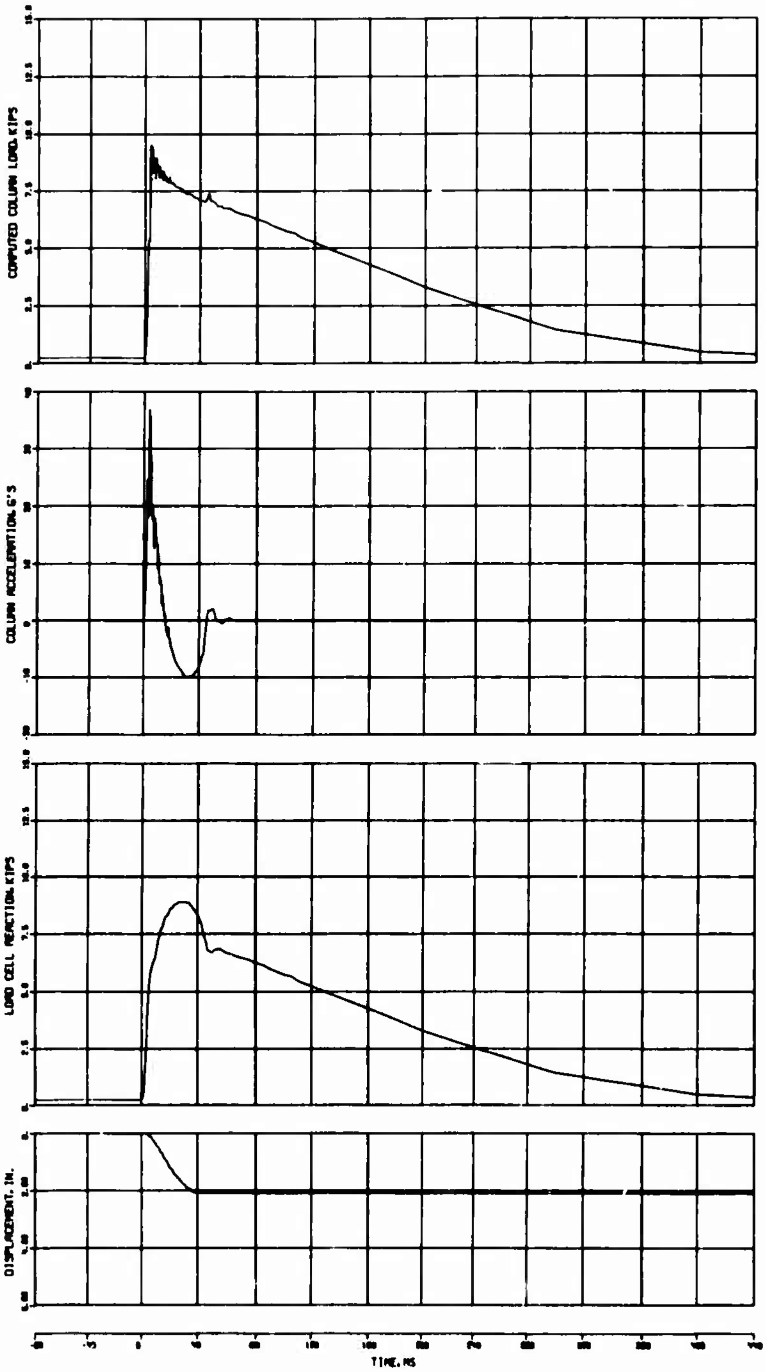
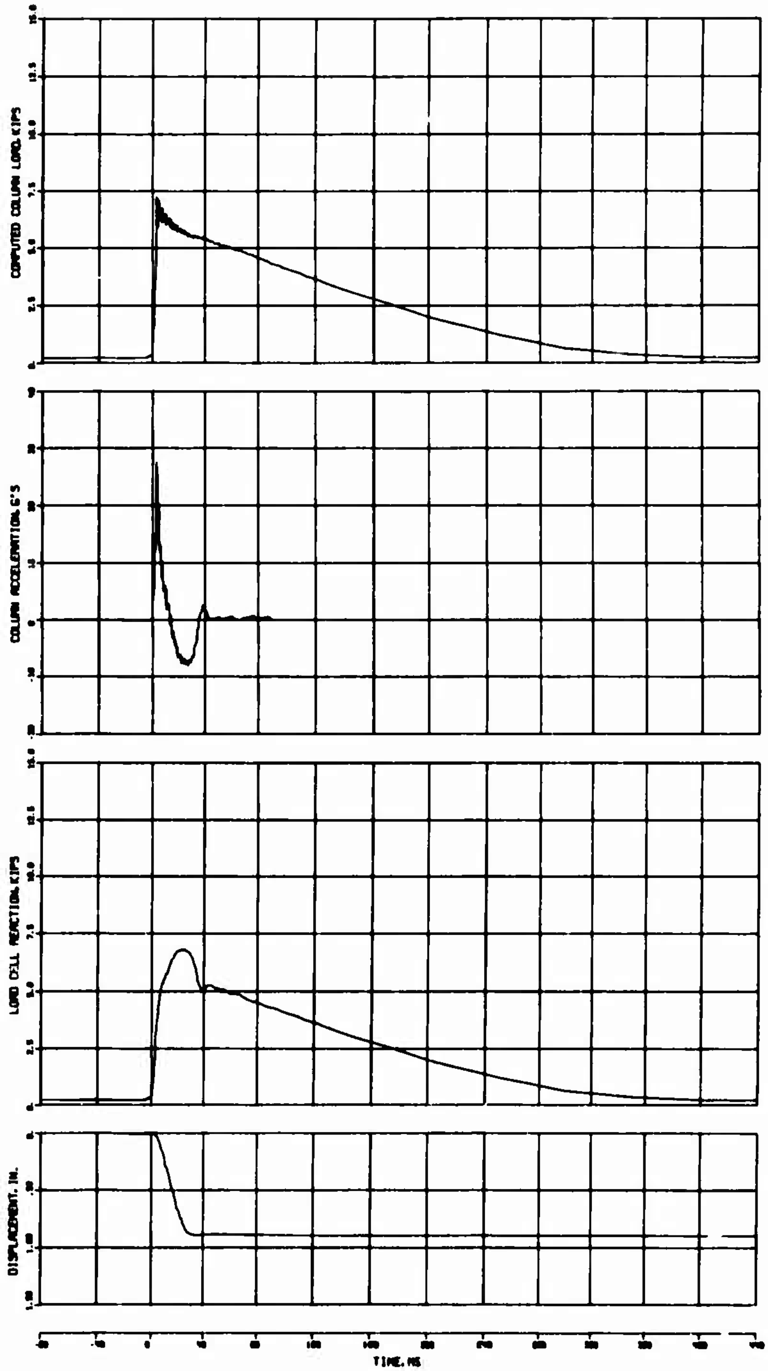


Figure D.2 Dynamic test 58-2.



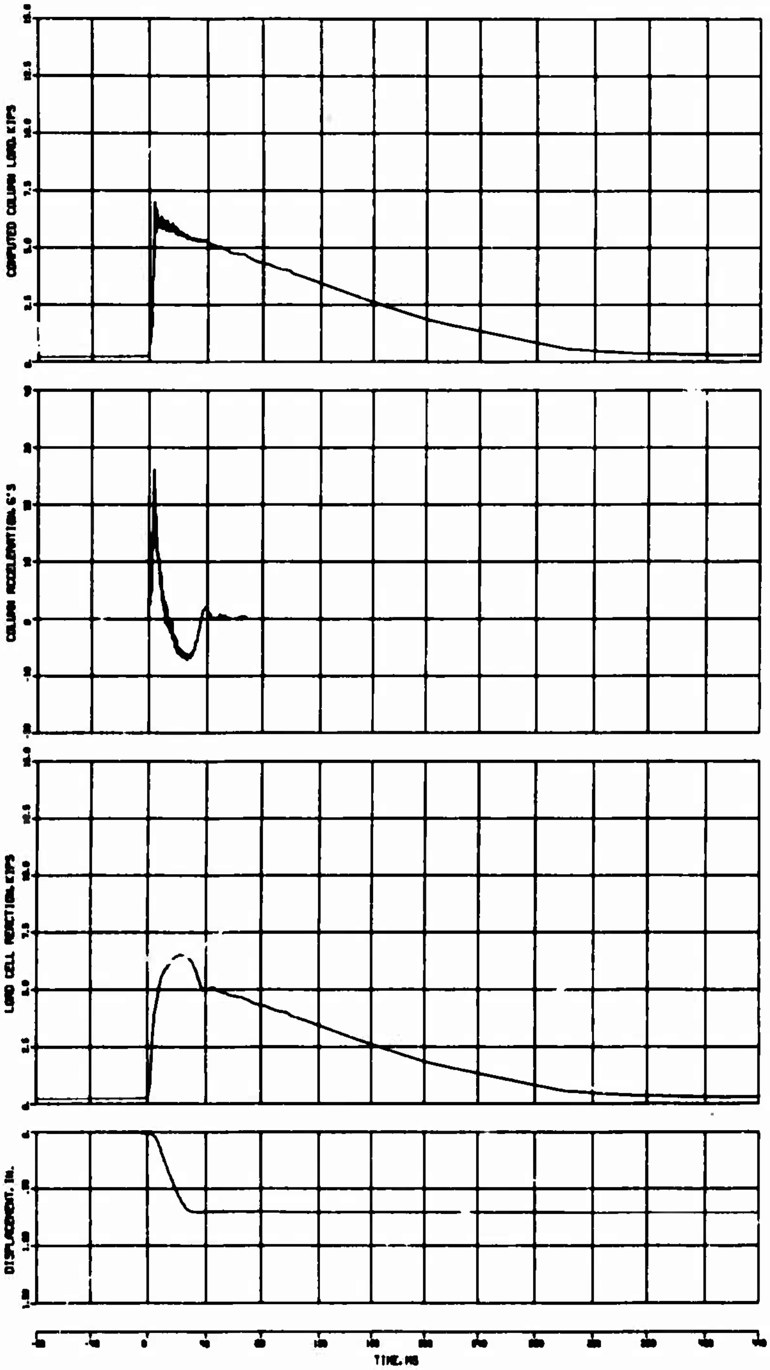
b = 8 IN.
D/b = 1
T_f = 150 KIPS/SQ FT

Figure D.3 Dynamic test 58-3.



b = 8 IN
D/b = 1
T_f = 125 KIPS/SQ FT

Figure D.4 Dynamic test 58-4.



b = 8 IN.
D/b = 1
T_f = 140 KIPS/SQ FT

Figure D.5 Dynamic test 58-5.

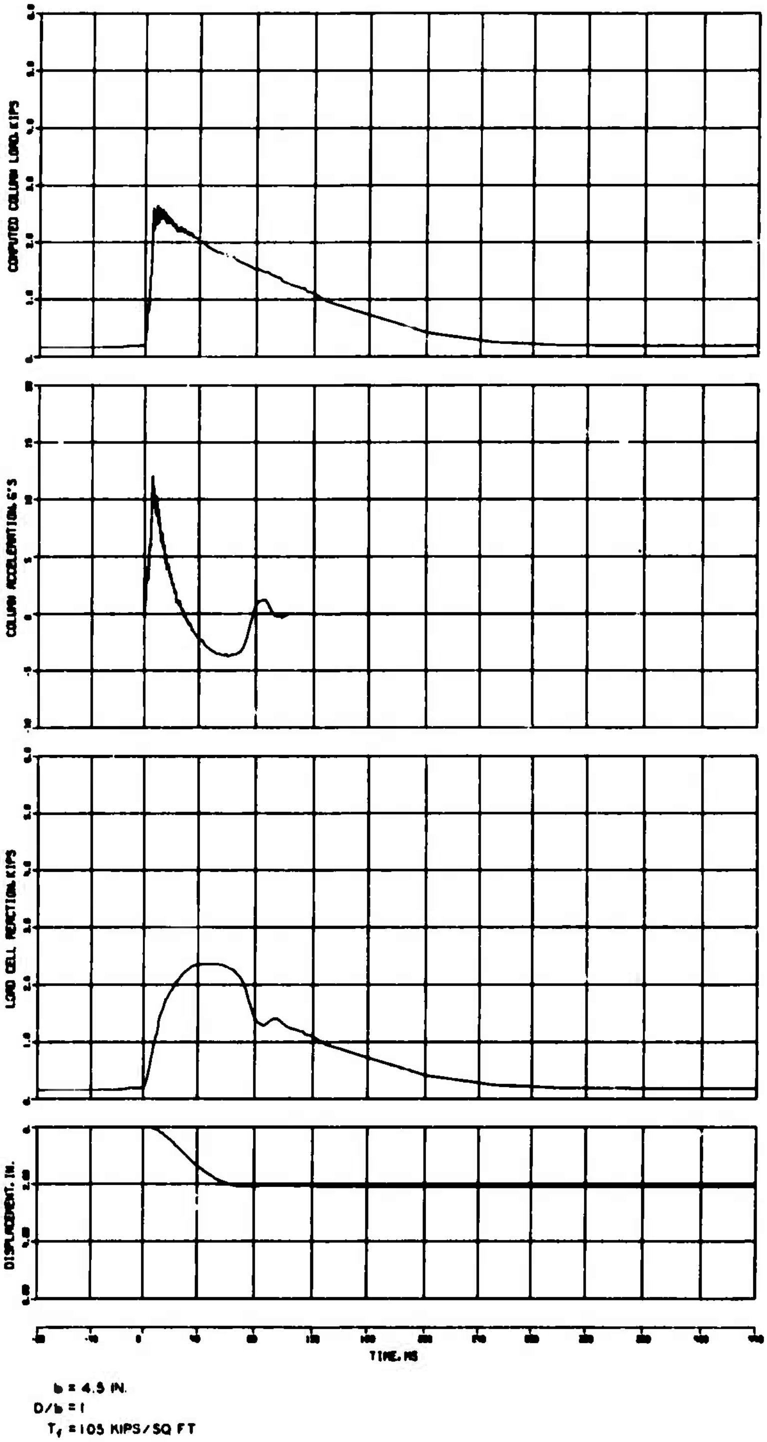
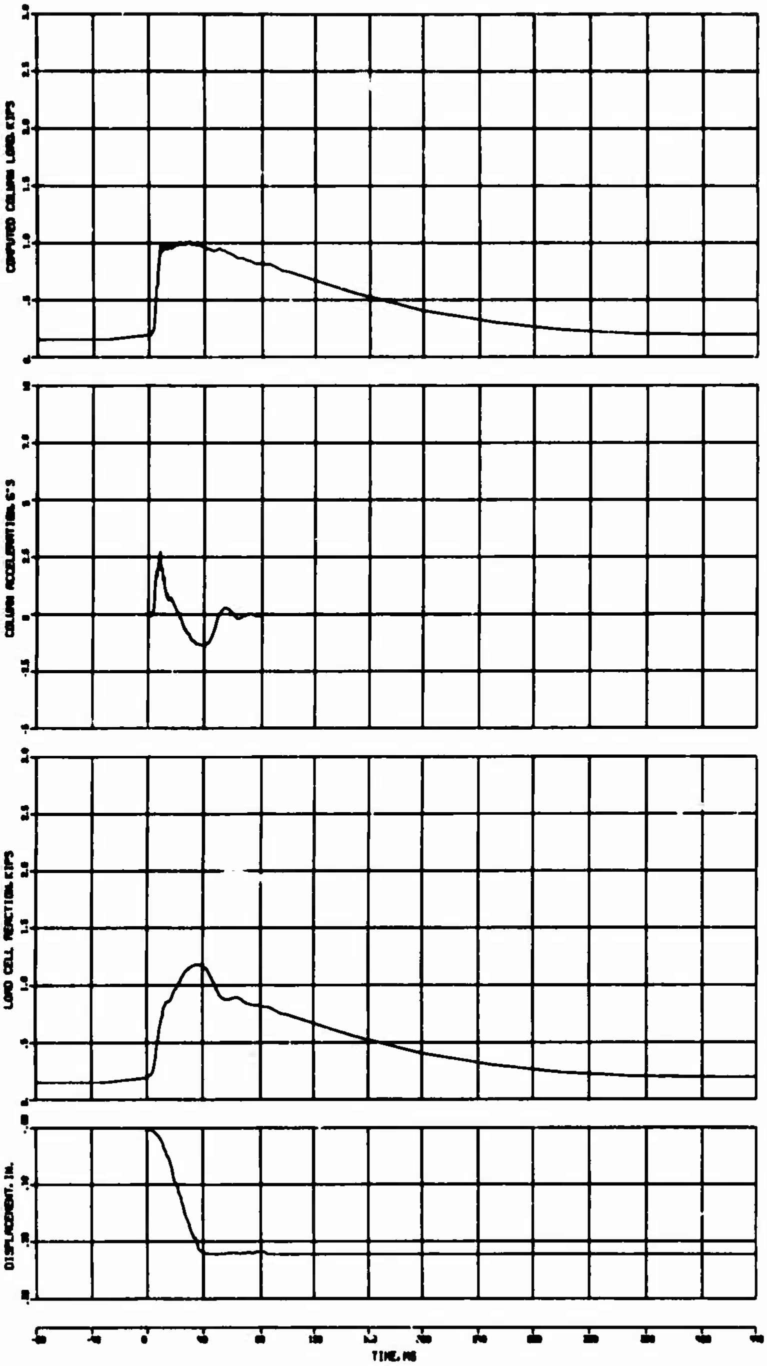
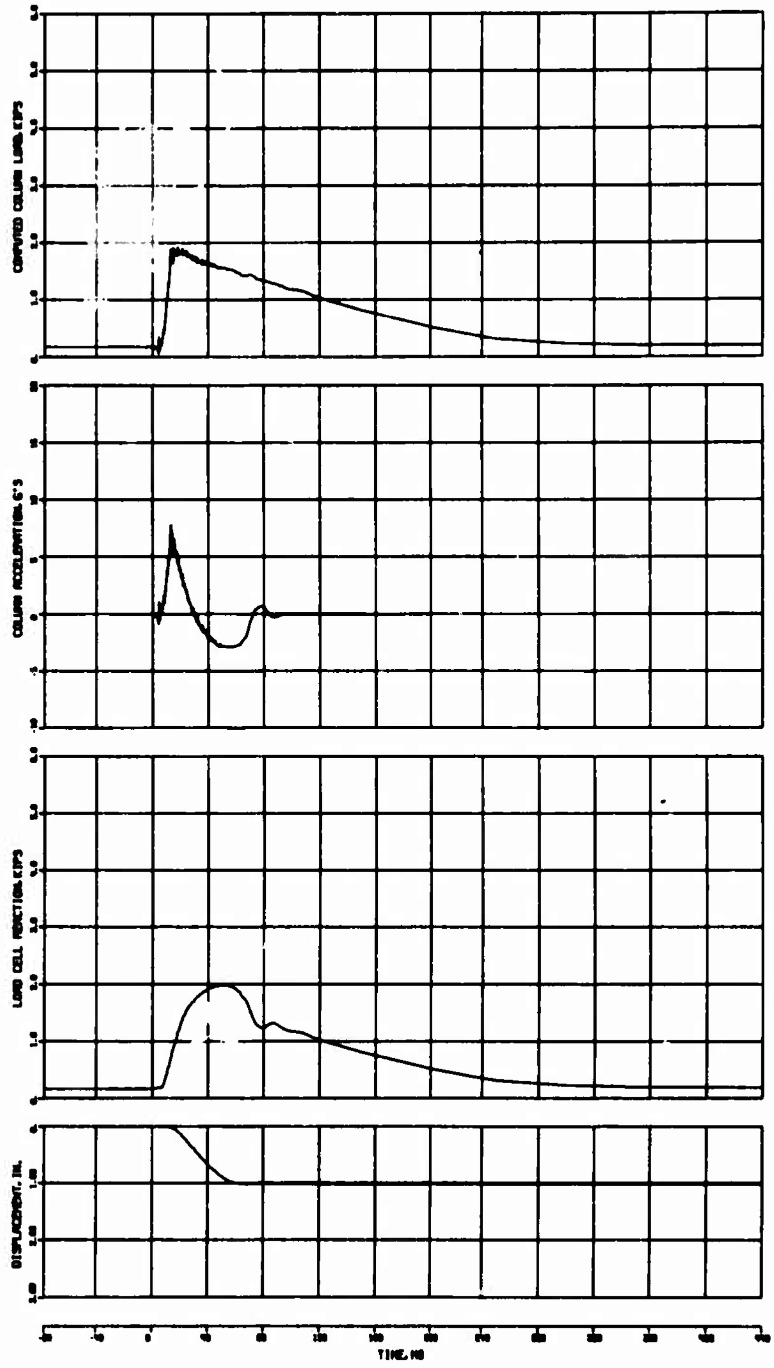


Figure D.6 Dynamic test 59-1.



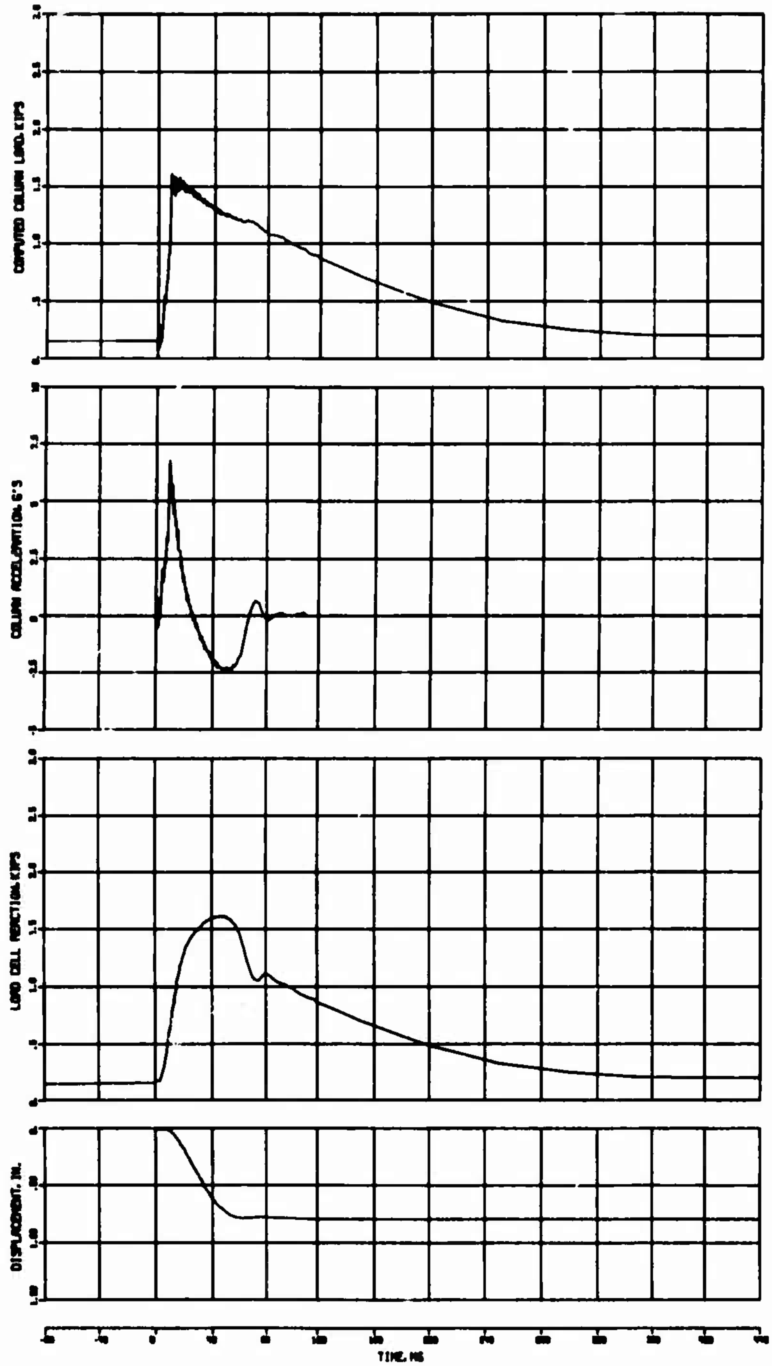
b = 4.5 IN.
D/b = 1
T_f = 1.10 KIPS/SQ FT

Figure D.7 Dynamic test 59-2.



b = 4.5 IN.
D/b = 1
T_f = 1.05 KIPS/SQ FT

Figure D.8 Dynamic test 59-3.



b = 4.5 IN.
D/b = 1
T_f = 1.00 KIPS/SQ FT

Figure D.9 Dynamic test 59-4.

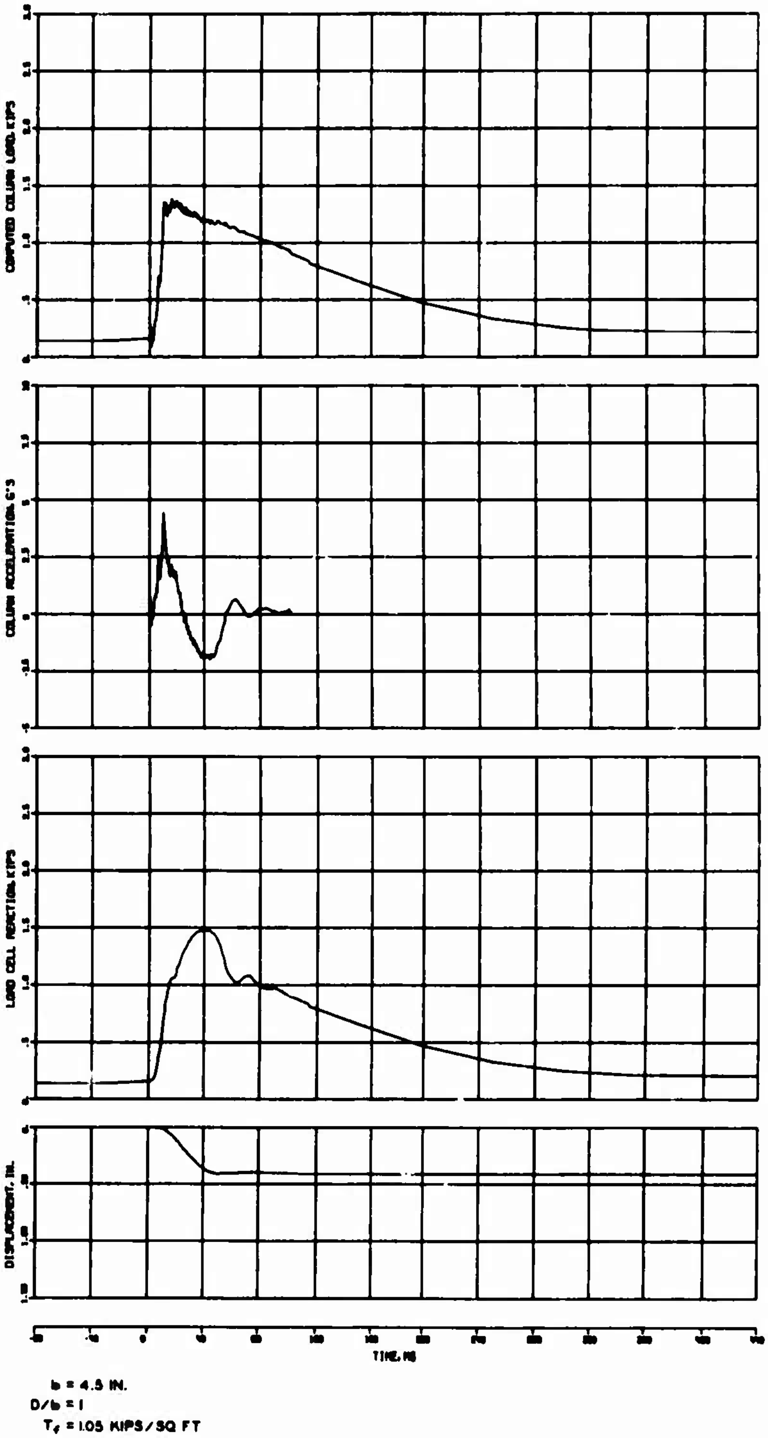


Figure D.10 Dynamic test 59-5.

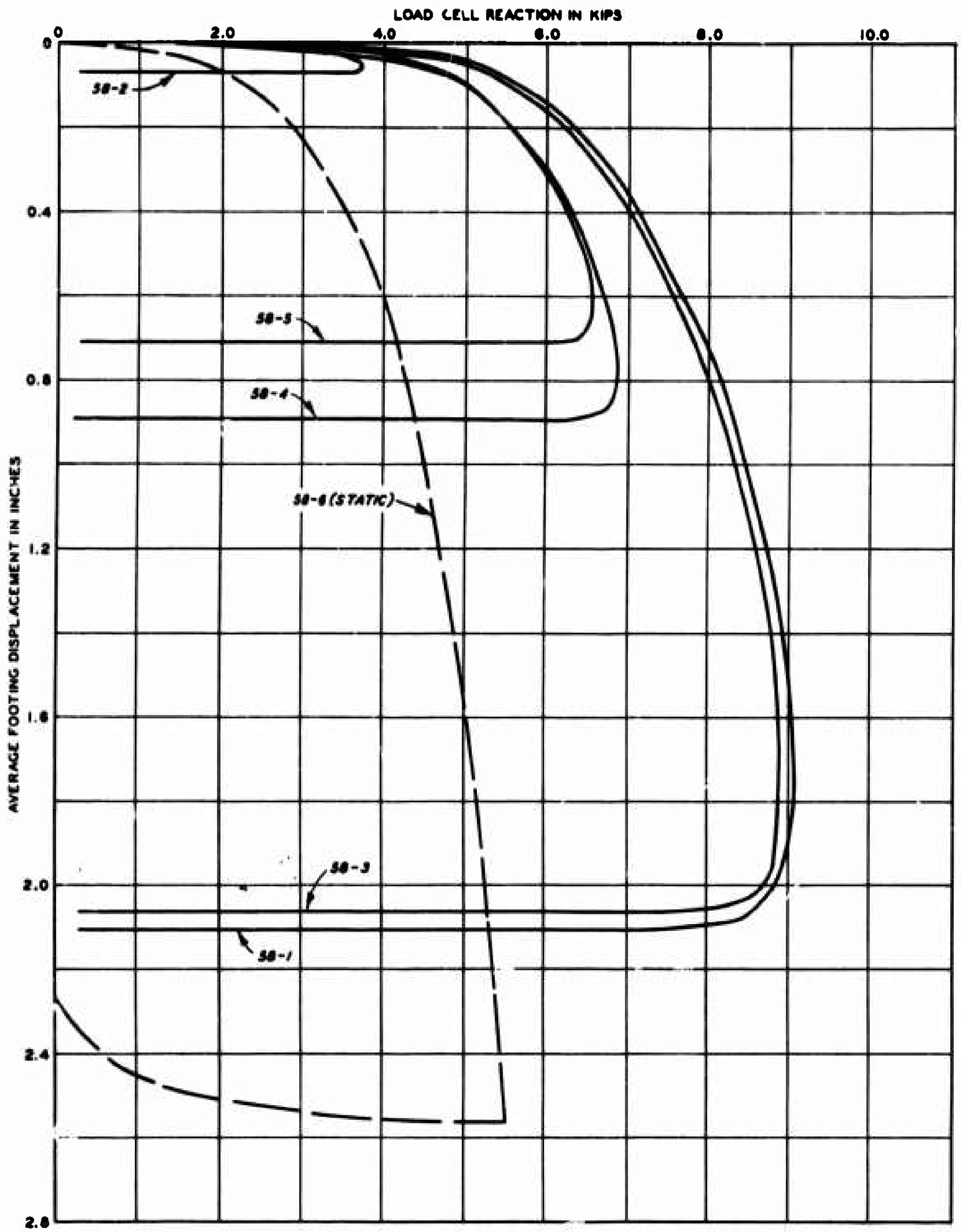


Figure D.11 Load-displacement curves, cart 58.

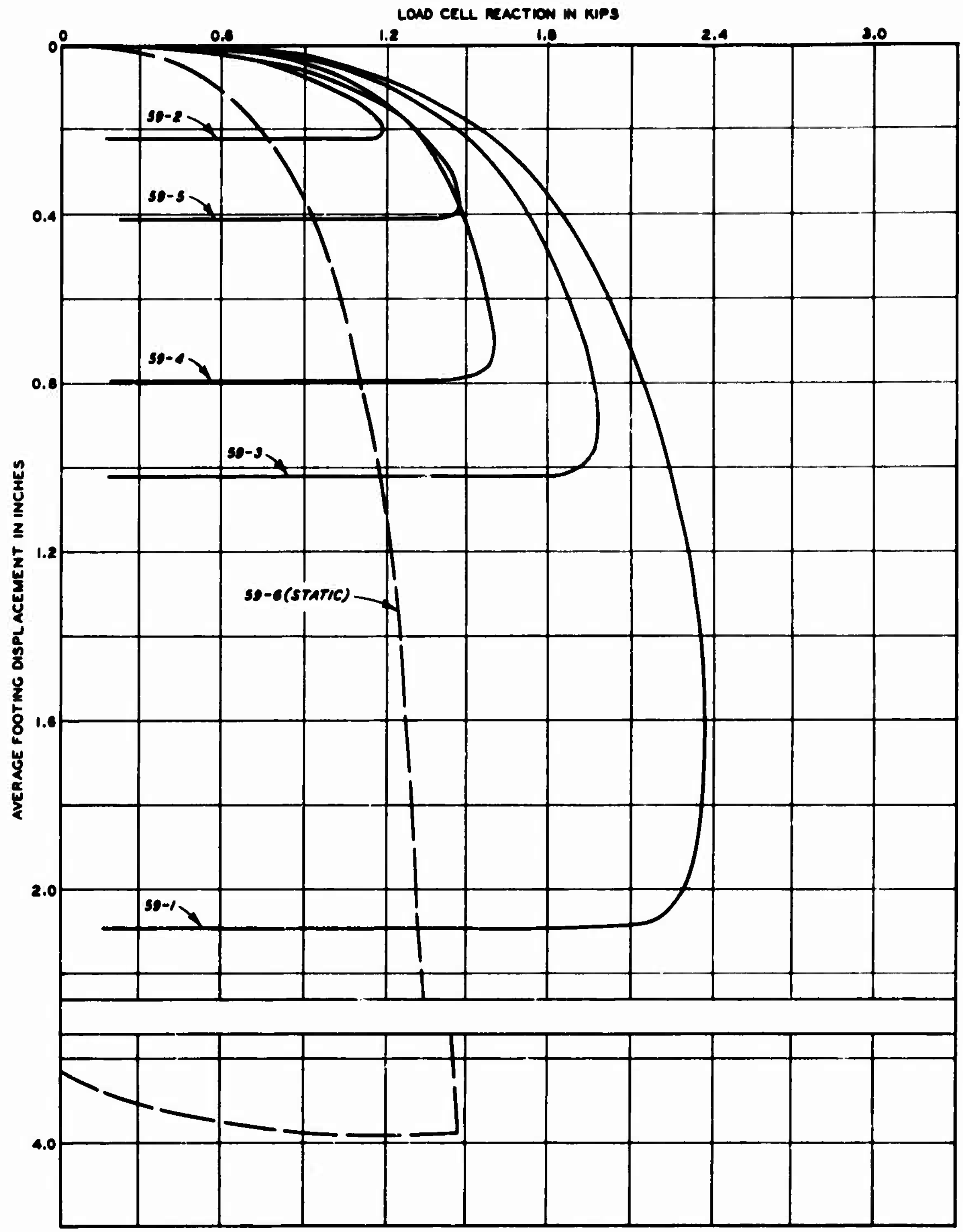


Figure D.12 Load-displacement curves, cart 59.

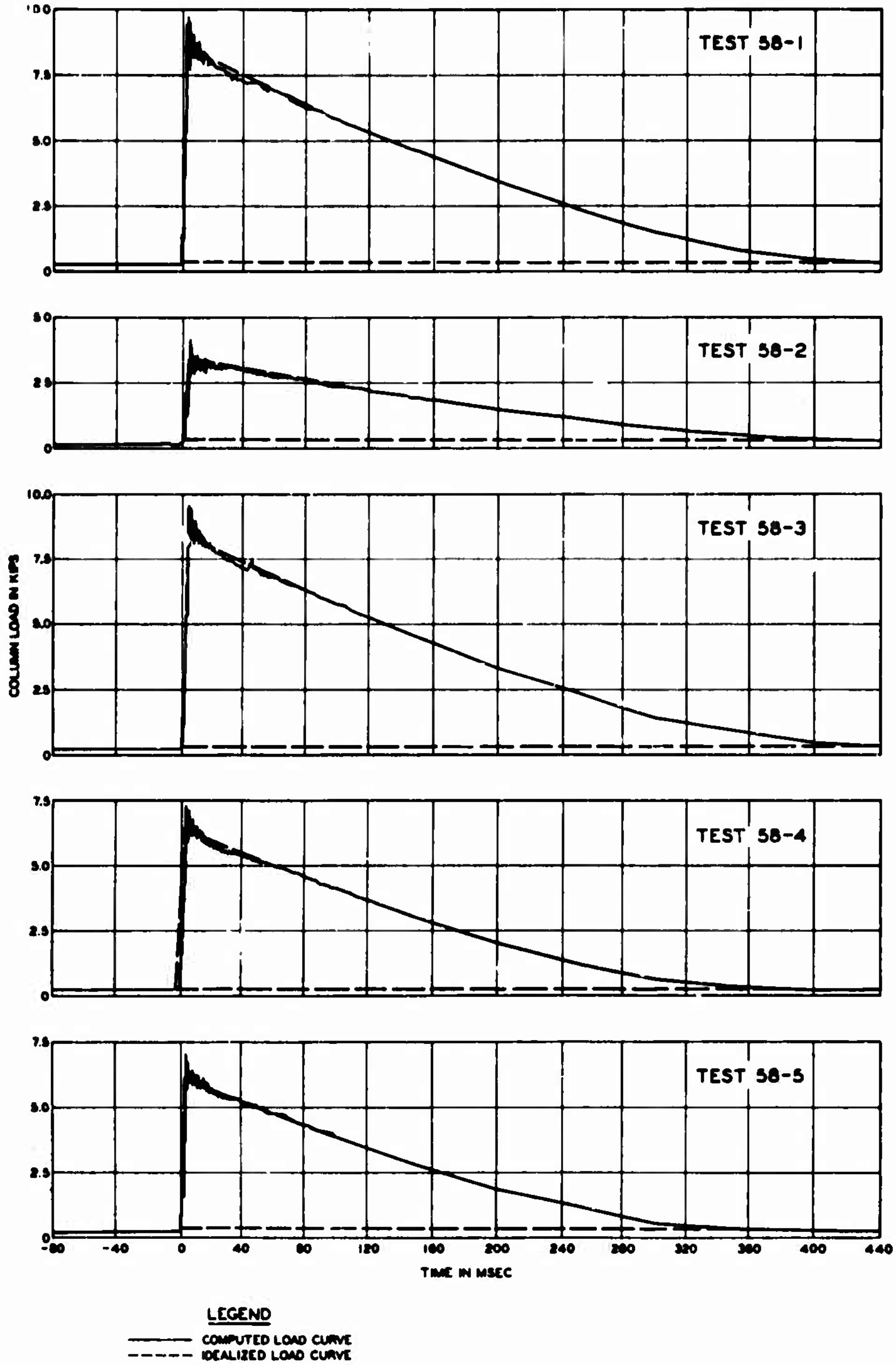


Figure D.13 Computed and idealized column load-time curves for dynamic tests, cart 58.

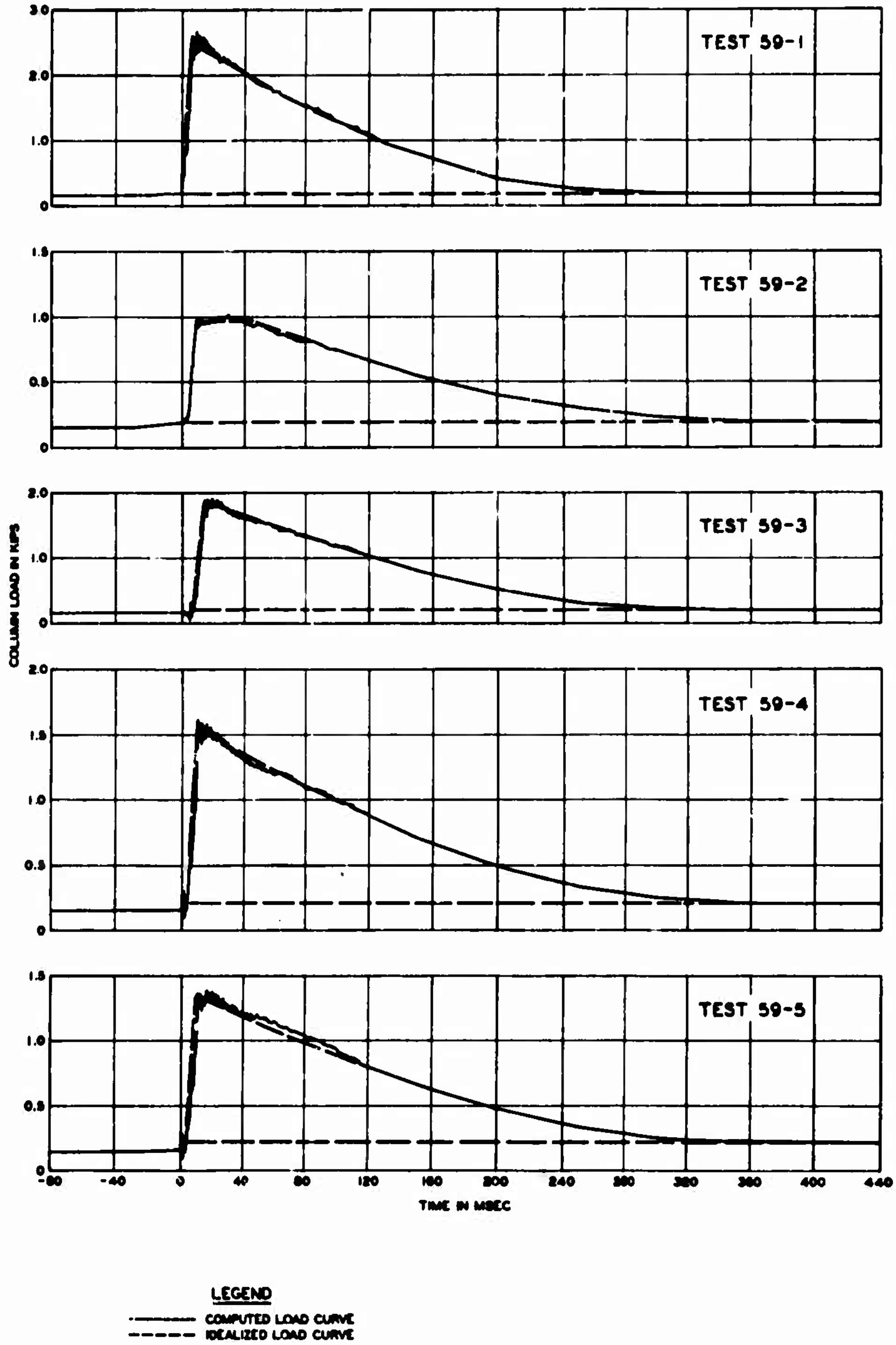


Figure D.14 Computed and idealized column load-time curves for dynamic tests, cart 59.

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13. ABSTRACT

The advent of nuclear weapons has resulted in the need for shallow-buried protective structures capable of surviving ground shock and associated loadings induced by high-intensity nuclear airblasts. The design of such structures requires a knowledge of the behavior of their foundations under rapidly applied transient loads. The purpose of this investigation was to determine if nondimensional relations developed previously by the application of similitude theory and dimensional analysis for dynamically loaded surface footings on clay also hold for dynamically loaded footings at shallow depth of burial and to determine the effect of shallow burial on the footing response. The results of the investigation showed that the nondimensional relations developed for surface footings on clay also hold for footings at shallow depth of burial. The effect of shallow burial on the response of dynamically loaded footings in clay is small and can be considered negligible for design purposes. ( )

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